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Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

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Journal of the
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WATER INTAKES IN THE DETROIT RIVER[a]

Eugene A. Hardin,[1] M. ASCE
(Proc. Paper 1592)

SYNOPSIS

A new water supply system will serve the suburban communities of the southeastern part of Wayne County, Michigan. Important features of this new system are a water intake in the Detroit River and a four mile long, 12-foot diameter raw water tunnel extending inland to a pumping station and purification plant. This system and other intakes of the Detroit River are described in this paper.

The Detroit River is a short strait connecting Lake St. Clair and Lake Erie. It is approximately 35 miles long from its beginning at Windmill Point (opposite Peach Isle) near the N.E. corner of the City of Detroit to its exit into Lake Erie at Pointe Mouilee (opposite Bar Point on the Canadian side). The International Boundary between the United States and Canada threads its way down the center of the stream, dividing the larger islands alternately between the two countries. See Fig. 1.

The southern half of the river's length is wide, varying from two to four miles in width, with islands and shallow "middle ground" between dredged navigation channels, all maintained to a minimum depth of 26 feet except Amherstburg Channel which is now 21 feet but is being dredged to 26 feet or more by the Great Lakes - St. Lawrence Seaway development.

The Detroit River carries all of the flow of the Great Lakes system above Lake Erie, averaging about 189,000 cfs, with an extreme variation from 99,000 to 242,000 cfs, a ratio of 1 to 2-1/2. Compare this with the flow in the Columbia River averaging about 200,000 cfs, at Bonneville, with a variation of 40,000

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a. Presented at a meeting of the ASCE, Buffalo, N. Y., June, 1957.

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cfs (minimum) to 585,000 cfs (maximum), a ratio of 1 to 15, and the flow in the Mississippi River averaging 482,700 cfs at Baton Rouge, with a variation of 73,700 cfs. (minimum) to 1,473,000 cfs (maximum), a ratio of 1 to 20. The recorded Detroit River level has varied less than 7 feet (between Elev. 570.34 and Elev. 577.17 above mean sea level) at the Fort Wayne (Detroit) gage, during the last 50 years. This variation in level is not so much from flow variations as from occasional weather conditions, heavy east winds from downstream causing the higher river levels and ice jams upstream and strong west winds causing the lowest levels; although there is a seasonal and a cyclical or phase variation. (See Hydrograph of Lake Huron and Lake Erie in Fig. 2). All of these data are regularly measured, recorded and charted by the U. S. Lake Survey division of the U. S. Corps of Engineers, which, among other many duties, also prepares and issues the Detroit River Navigation Chart.

The Detroit District Engineer's office of the U. S. Corps of Engineers maintains the navigation channel depths and widths and has supervision of all construction work and issues permits to construct, excavate or dispose of material in the river on the U. S. side of the International Boundary. Its Canadian counterpart is the Canadian Department of Public Works. The U. S. Coast Guard maintains the Aids to Navigation and safety features on the U. S. side of the International Boundary as does its counterpart, the Canadian Department of Transport on the Canadian side.

The east bank of the Detroit River is in the province of Ontario, Canada, and along this bank are located, from north to south, the cities of Riverside-Tecumseh, Walkerville, Windsor, Sandwich, East Sandwich and West Sandwich, Ojibway, LaSalle, River Canard and Amherstburg. Its west bank is in the State of Michigan, U. S. A., and practically all in Wayne County. Along this bank are situated, from north to south, the cities of Detroit, River Rouge, Ecorse and Wyandotte, and the villages of Riverview, Trenton, Gibraltar and East Rockwood. All of these municipalities obtain their water supplies in some manner from the Detroit River. However, there are now only four public water supply intakes in the Detroit River.

The approximate locations of present water supply intakes in the Detroit River are indicated by the circled numbers on a reproduction of the Detroit River Navigation Chart given in Fig. 3.

Existing Water Intakes on the Detroit River

The existing public water supply intakes are as follows:

1. The Essex Border Utilities District intake consisting of duplicate submerged timber cribs for the water works supplying Riverside-Tecumseh, Walkerville, Windsor, the Sandwiches, Ojibway and LaSalle.
2. The City of Detroit intake—a lagoon and masonry pier intake supplying Detroit, River Rouge, Ecorse, Riverview, Trenton and Gibraltar, along with several other cities and areas further inland.
3. The City of Wyandotte intake—a submerged timber crib supplying Wyandotte and some surrounding areas.
4. The City of Amherstburg intake—a pier-head intake supplying Amherstburg and some surrounding area, as well as industrial water to the Brunner Mond Canada, Limited, alkali manufacturers.

Industrial water supplies are also taken from the Detroit River by large manufacturing plants at the points numbered 5 to 13, as follows:

5. Solvay Process Division of Allied Chemical & Dye Corporation.
6. Great Lakes Steel Corporation (6a, 6b and 6c).
7. DuPont Company.
8. Wyandotte Chemicals Corporation (8a and 8b).
9. Michigan Alkali Company.
10. Pennsalt Chemical Corporation (10a and 10b).
11. Detroit Edison Company (11a, 11b, 11c and 11d).
12. The Hiram Walker Distillery in Walkerville, Ontario.
13. The Ford Motor Company's River Rouge plant which uses what is essentially Detroit River water in large quantities for industrial purposes but takes it from the old channel of the Rouge River. The dredged ship channel of the Rouge delivers the flow of the Rouge River to the Detroit River at the south end of Zug Island, approximately one mile downstream from its natural mouth, permitting the old channel of the Rouge to reverse itself and bring Detroit River water to the Ford intake.

A canvass of users of water from the Detroit River produced the summary of information shown in Table 1.

The general design data of the Detroit River Water Supply intakes are compared in Table 2.

It will be noted that these intakes are of four basic types:

Public Supplies	(A Submerged crib) (B Masonry pier)	Located out in the river
Industrial Supplies	(C Pier-head inlet) (D Open Channel or Conduit)	Located at the river's edge

Public Water Supply Intakes

Submerged crib type intakes are used by the Essex Border Utilities District and by the City of Wyandotte, with intake pipes ending in submerged cribs of heavy timbers, latticed to form pockets for large stone which weight them down, and protected by stone fill placed around them. These intakes are comparable, but there are some differences in detail to accommodate them to their sites. For example, the Essex Border Utilities cribs are located about 500 ft. offshore on the Canadian side of the navigation channel at the bottom of the slope. At this break in the slope the intake pipes enter the cribs from the bank side and above the crib. The plane of the pipe opening is thus practically vertical and less liable to the entrance of material and the pipe is not under the crib. However, the location of the Wyandotte crib on the bottom of a dredged channel in the "middle ground" makes it necessary to have the intake pipe buried below the channel with a tee riser opening inside the crib. In this case, the plane of the opening is horizontal and subject to the entrance of material falling from above. A protective screen was used on top of the old crib and the new crib has a timber grid above the pipe opening. Fig. 4 shows the old and the new Wyandotte cribs, which are typical timber cribs.

The Essex Border and the Wyandotte intakes are subject to many similar conditions, and have had some common difficulties, such as clogging with ice; although Wyandotte has had less ice trouble.

At Wyandotte, due to damage to the old intake pipe line from dragging anchors, the engineers for the new intake made some practical studies of the problem and established a minimum of 8 ft. of cover as a safe depth of pipe where crossing a ship channel. The old intake line had 5 ft. of cover or less.

A feature of the Essex Border Utilities intakes is provision for backflushing with water if a crib should become clogged with ice. Since the cribs are in the main flow channel of the river, when the backflushing loosens the ice, most of it is carried away by the river current, although some enters the pipe.

Examination of the old intake crib by the Essex Border Utilities engineers when the new intake was built found it to be in excellent condition after almost 30 years' use. The only fault found was a slight erosion of the river bed under the upstream edge of the crib and the displacement of a small amount of the rock ballast, possibly by backflushing. This was corrected by placing additional rock on top and around the base of the crib.

The successful and reasonably trouble-free operation of submerged crib intakes and their relatively low cost recommend them, where the conditions are favorable, if designed on the empirical lines established by past successful practice.

The masonry pier type intake is typified by the City of Detroit's intake, built in 1930-32.

The general construction of the City of Detroit intake is shown in Fig. 5. It is located at the end of an artificial lagoon constructed at the upstream end of Belle Isle. The lagoon provides a stilling basin in which heavy sediment and debris will settle and needle ice or frazil will rise and form an ice sheet or adhere to the ice sheet, if already formed. This has proven very successful in operation since there has been no trouble at all in this intake from any of these causes.

A photograph of the Detroit Intake Structure taken soon after completion is shown in Fig. 6.

A pier-head type intake is used at Amherstburg. Its general design is sketched in Fig. 7. One-half of this pier intake supplies a purification plant for the potable public water supply of Amherstburg, while the other half of this pier provides industrial water to Brunner Mond.

Industrial Water Supply Intakes

The industrial water supply intakes along the Detroit River are in general of two basic types:

Type C - A pier-head inlet located at the edge of the river, with inlet ports in a masonry pier supported on piling, with vertical racks across the face and traveling screens in a chamber behind, from which a conduit connects to the plant, as at the Pennsalt Chemical plant shown in Fig. 8, and at the Great Lakes Steel Plant shown in Fig. 9; and

Type D - An open channel or conduit leading from the river's edge to a battery of racks and traveling screens and thence through large conduits to the points of use, typified by the Detroit Edison Plant intakes, indicated more or less diagrammatically in Fig. 10.

The design capacities, sizes and velocities of the industrial water supply intakes are tabulated in Table 2; indicating the general use of less submergence and higher velocities in the industrial water intakes than in the

public water supply intakes. Yet these plants report none or very little trouble from ice or obstructions. This freedom from ice troubles may be attributed to the warmer winters of the last 20 years and to improved provisions for coping with ice.

Since the industrial supplies are not concerned with potable quality they can take the more polluted water from along the edge of the River. But for a public water supply it is a far different problem, and sanitary engineers go to great lengths to locate an intake where it will take in the best quality of water reasonably obtainable.

Proposed Intake for the Wayne County Water Supply

A large public water supply intake to be built in the Detroit River has been designed to provide raw water to a pumping and purification plant which will serve the southwestern part of Wayne County, Michigan; approximately one-half the area of the county. An intensive study was made of the data accumulated over many years on the flow and behavior of the river, including periodic sampling and testing of the water quality at many points. This was supplemented by a year of continuous sampling, gaging and analysis by a team of exceptionally capable sanitary engineers; and a zone of relatively pure water was found to exist in the main flow of the stream between the threads of 40,000 cfs flow, or in the central half of the river flow. This is easily accepted when one realizes that the Detroit River is not a "garden variety" of stream. It differs from the normal river in the following respects:

1. Its flow rates are relatively uniform;
2. Its surface level is relatively constant;
3. Due to the enormous storage in the lakes, it has no floods;
4. It has a uniformly high velocity in its main flow channels at all times; and
5. The maintained navigation channels and the islands keep the flow in definite paths.

Consequently, we have here a river of quite definitely known behavior, and completely controlled by the very nature of its environment.

The classic study of pollution in the Great Lakes waters, made by the Joint Commission on the Pollution of Boundary Waters in 1914, established that the pollution follows the shore lines and diminishes rapidly toward the center of the stream. This has been verified time and again by all who have studied the Detroit River.

It is well known also that the ten mile reach in the mid-length of the river is least affected by storm disturbance in the lakes above and below. For example, an easterly wind may blow in water from Lake Erie and back up some pollution a few miles upstream to Grosse Isle, but here the river is narrowed down by the island and the large flow rate of the river maintains its cleansing current, which becomes stronger as the flow channel narrows upstream.

So, from the standpoint of water quality, an intake could be placed at any feasible point in the main flow channel between the middle of Grosse Isle and the head of Fighting Island, the location being determined in general by the site adopted for the pumping and purification plant. Fortunately, the main flow channel of the stream meanders slightly in this portion of the river so that it crosses to the east side of the navigation channel for a mile or so and

then crosses to the west side, and so on. Consequently, there are sites for an intake in deep water in the main stream flow and outside of the designated navigation channel which is straight.

The plant site, in turn, was determined in general by the load center of the service area and specifically by the available undeveloped property that could be acquired.

The adopted intake site, as shown in Fig. 3, is almost due east of the chosen plant site so that the raw water tunnel may be built under Riverbank Avenue in a practically straight line to the shore shaft and then, at a slight angle, from the shore shaft to the intake site in a straight line, tunneled in rock across the river at a depth of 100 ft. or more below the bottom of the river channel.

The natural river flow channel at this point meanders to the Canadian side of the navigation channel leaving no feasible location on the U. S. side. Consequently, the intake here will be located in Canadian waters. At the request of the Lake Carriers' Association, and approved by the regulating authorities of both the U. S. and Canada, the intake structure is to be located with at least 200 ft. of clearance between it and the east side of the navigation channel. Since it will be in the main flow current of the river, it will be of streamlined, oblong shape with its narrow edge in the path of the stream to present as little obstruction to flow as possible, and also to provide for water inlet ports in the long sides so that the direction of flow entering the ports will be normal to the stream flow.

The river velocity here is approximately 3 miles per hour which gives a relatively large velocity past the ports compared to the small velocity into the ports. This will tend to carry objects that may be floating or suspended in the stream past the ports before they are turned into them.

It will be noted that the adopted location of the intake structure is more or less ideal from the viewpoint of its sanitary quality. It is shielded by Fighting Island from the pollution of the Canadian shore, and pollution from the U. S. shore is prevented by the high velocity in the river channel from crossing over to the intake. Practically all of the pollution in this reach of the river passes down the Trenton channel along the U. S. shore and down the back channel along the Canadian shore, and the existence of Fighting Island on one side and the shallow "middle ground" on the other practically assures that this condition of river flow will remain unchanged.

Design Criteria

480 Million Gallons Daily was adopted as the ultimate flow capacity for which to design this intake system, capable of supplying a population of one and one-half million people and the great expansion of industrial development expected in the area within the next 50 years.

Fig. 11 shows the general design of the proposed Wayne County Intake. The intake structure was designed to the following criteria:

1. Maximum velocity in ports and through racks—0.5 ft. per sec.
2. Structure to be founded on solid rock in the bed of the river and to extend as a pier 5 ft. above the highest water level of the river.
3. Port openings to be on the two long sides in two tiers, the lower tier with sills five feet above the river bottom and the upper tier with tops five feet below the lowest water level.
4. Port openings to be provided with means for closing off either side or

either tier of ports, if river conditions require; for example, the top tier may be closed in winter against icing conditions and the lower tier closed in summer during dredging operations.

5. Trash racks to be provided inside the structure to prevent the entrance into the tunnel shaft of material which might clog the tunnel or damage the pumps at the plant.
6. Trash racks each to be in a separate channel serving two ports, which will be in vertical alignment.
7. Stop plank grooves to be provided so that any rack may be isolated for inspection and repair.
8. Large openings to be left in the top floor slab above the racks and stop gate grooves for access; these openings to be covered with removable concrete slabs.
9. Navigation lighting and signals to be provided to satisfy the requirements of the U. S. Coast Guard and the Canadian Dept. of Transport.
10. Design for the following river levels (the average of recorded levels at the Gibraltar and Ft. Wayne gages):

Maximum water level—Elev. 576.8
Minimum water level—Elev. 568.8
Mean low water datum-Elev. 571.8

Method of Construction

Since the design and structural details are influenced a great deal by the possible methods of construction that can be used, considerable study was given to this problem; taking into account the river depth, velocity of the current, depth in river bottom clay to the rock surface, and the river traffic, weather conditions, etc. Finally, two alternative methods were adopted as the most likely to be used in construction of the intake structure.

Either - To build inside of a peripheral, self-supporting cellular type cofferdam;
Or - To construct a hollow-steel-walled open caisson on the shore, float to the site and sink into place on the bed rock;
And - Complete the internal structure in the dry in both cases after placing a heavy concrete seal across the bottom.

Two alternate designs based on these methods were drawn up and bids received on either type. The low bidder based his bid on the caisson method of construction.

Fig. 12 is an artist's rendering of the intake after completion, looking from Fighting Island toward the U. S. shore.

Wayne County Intake Tunnel

The intake tunnel will be in solid rock at from 100 ft. to 150 ft. below the bottom of the river channels, as shown in Fig. 13.

It will be 12 ft. inside diameter, concrete lined, and about a mile long, with intake shaft at the outboard end and a shore shaft at the river bank.

It may be of interest that a comparative study of a river conduit of precast sections laid in a trench dredged across the river above the rock indicated that such construction would cost considerably more than to tunnel in the rock. This was due to the deep cut through the "middle ground."

The land tunnel will also be 12 ft. inside diameter, concrete lined, shield driven in the soft blue clay typical of the Detroit area, and will extend inland about 3 miles to the pumping and purification plant site.

Cost

Bids were received on September 11, 1957, and contracts were awarded to the lowest bidders on each of the three main sections of the work as follows:

Section A -	Intake structure, shaft and approximately 200 feet of tunnel on the Canadian side of the International Boundary	$2,825,250
Section B -	Shore shaft and river tunnel (in rock) on the U. S. side of the International Boundary	$2,359,400
Section C -	Land Tunnel in clay	$5,195,850
	Total	$10,380,500

The total cost compared favorably with preliminary estimates made during the preceding 18 months.

Operation and Maintenance Facilities

One of the bug-a-boos of water intakes in the Great Lakes and connecting streams is frazil or needle ice, which can be drawn into an intake along with the water. Here it will anchor to the colder metal gate frames and trash racks or screens and restrict or close off the openings. Troubles from frazil are reduced or eliminated in lake intakes by deep submergence of the crib or inlet ports, 30 ft. being the desirable minimum (30 ft. is also below the draft of most boats that may be in the area); by low velocity in the inlet ports; and by locating in surface stilled water, such as in a lagoon or under a sheet of ice.

But an intake located in 30 ft. total depth of water in the main current of a rapidly flowing stream beside a dredged navigation channel, far from shore and in foreign waters, as in the case of the Wayne County intake, cannot have or develop all of these boons. It is therefore incumbent on the designer to incorporate the best elements of good practice he can, to provide as simple an arrangement as possible to facilitate maintenance and operation, and to provide means to cope with the unavoidable difficulties that may occasionally arise.

On the Wayne County intake pier there will be provided:

1. A sufficient supply of stop plank gates to shut off any or all ports or rack chambers,
2. An extra set of rack bars,
3. A small gas powered automobile crane to handle removable slabs, stop gates, racks, etc.;
4. An electric power generator, a small air compressor, and a pressure water pump, all gas engine driven,
5. Propane gas supply facilities for fueling the gas engine driven equipment and for heating, cooking and maintenance facilities,
6. Flood lights,
7. Davits for handling small material from boat to pier, and
8. A two-way pier-to-shore radio intercommunication unit.
9. These materials and equipment will be stored or installed in the two small buildings at the ends of the structure, in which also will be

provided space for a small maintenance shop and laboratory and shelter quarters for workmen, with subsistence facilities and supplies.

Since it is expected that the racks will be the first to clog with ice, provisions will be made for applying electric heat to the rack bars, (or possibly vibrators), and for applying jets of air or water or steam under the racks. It may take some experimentation for a year or two of actual operation to find the best preventative or cure for ice troubles here, and so discussion at this time would be purely speculative.

ACKNOWLEDGEMENTS

The writer wishes to acknowledge, with thanks, the valuable assistance of the engineers of the cities and industries in supplying the data on their water supply intakes, which made this paper of general interest.

EXISTING WATER INTAKES — DETROIT RIVER

TABLE I

	USE OR OWNER		DATE BUILT	DESIGN CAPACITY (m g d)	TYPE OF INTAKE	DEPTH OF WATER ABOVE INTAKE
	PUBLIC WATER SUPPLIES:					
(1)	ESSEX UTILITIES DIST.	Old	1925	63	Submerged crib	27 to 29 ft.
		New	1953	84	Submerged crib	28 to 30 ft.
(2)	DETROIT		1929-32	900	Masonary pier	10 to 12 ft.
(3)	WYANDOTTE		1950	25	Submerged crib	20 to 25 ft.
(4)	AMHERSTBURG		1918	10	Pier head	5 to 7 ft.
	INDUSTRIAL WATER SUPPLIES:					
(5)	SOLVAY PROCESS CO.					
(6)	GREAT LAKES STEEL	Steel Plant (a)	1930	120	Pier head	0 to 5 ft.
		Blast Furn. (b)	1933	50	Pier head	0.5 to 6 ft.
		Blast Furn. (c)	1950	50	Pier head	0.3 to 6 ft.
(7)	DUPONT CO.		1927	3 to 6	Pier head	1 to 5 ft.
(8)	WYANDOTTE CHEMICAL CO.	North Works (a)	1924	86	60" pipe	4.5 to 7 ft.
		South Works (b)	1930	65	Pier head	0 to 1 ft.
(9)	MICHIGAN ALKALI CO.		1930	94	7' x 12.5' conduit	0 to 2 ft.
(10)	PENNSALT CHEMICAL CO.	North (a)	1910-15	43	Pier head	3 to 7 ft.
		South (b)	1916-48	75	Pier head	3 to 7 ft.
(11)	DETROIT EDISON CO.	Conners Cr. (a)	1915	945	Open channel	—
		Delray (b)	1929	886	Open channel	—
		River Rouge (c)	1956	1300	Covered channel	4 ft.
		Trenton Ch. (d)	1924	1270	Open channel	—
(12)	HIRAM WALKER DISTILLERY	Old	1900	10	48" pipe	25 to 30 ft.
		New	1955	10	30" pipe	1 to 6 ft.

GENERAL DESIGN DATA — DETROIT RIVER INTAKES

TABLE 2

	SUPPLY		DATE INSTALLED	DESIGN CAPACITY (m.g.d)	INTAKE CONDUIT		INTAKE STRUCTURE						
							PORTS				RACKS		
					Diam or Dimen.	Veloc (ft./sec)	Type	Number	Area (sq. ft)	Veloc (ft./sec)	Number	Area (sq. ft)	Veloc. (ft./sec)
	PUBLIC WATER SUPPLIES:												
(1)	ESSEX UTILITIES DIST.	Old	1925	50	48"	6.2	A	1	28.3	2.74	–	–	–
		New	1953	80	54"	7.8	A	1	50.3	2.47	–	–	–
(2)	DETROIT		1929-32	900	186"	7.4	B	10	1615	0.86	20	2000	0.7
(3)	WYANDOTTE		1950	25	42"	4.0	A	1	9.6	4.02	1	54	0.7
(4)	AMHERSTBURG		1918	10	–	–	C	2	50	3	2	88	1.8
	INDUSTRIAL WATER SUPPLIES:												
(5)	SOLVAY PROCESS CO.												
(6)	GREAT LAKES STEEL	Steel Plant (a)	1930	120	–	–	C	3	240	0.77	3	200	0.9
		Blast Furn. (b)	1933	50	–	–	C	1	104	0.74	1	85	0.9
		Blast Furn. (c)	1950	50	48"	6.16	C	1	62	1.25	–	–	–
(7)	DUPONT CO.		1927	3 to 6	–	–	C	1	48	0.1 to 0.2	1	24	0.2 to 0.4
(8)	WYANDOTTE CHEMICAL CO.	North Works (a)	1924	86	6.5'x8.5'	2.4	D	1	28	4.7	–	–	–
		South Works (b)	1938	65	6'x8'	2.1	C	3	63	1.6	1	65	1.5
(9)	MICHIGAN ALKALI CO.		1930	94	7'x12.5'	1.7	D	2	85.5	1.7	1	68	2.14
(10)	PENNSALT CHEMICAL CO.	North (a)	1910-15	43	4'x4'	4.16	C	2	25	2.65			
		South (b)	1916-48	75	7'x7'	2.4	C	4	50	2.34			
(11)	DETROIT EDISON CO.	Conners Cr. (a)	1915	945	200'x8'	1.0	D						
		Delray (b)	1929	886	100'x11'	1.25	D						
		River Rouge (c)	1956	1300	118.6' x 8.6'	2.0	D	1	1000	2.0	20	900	2.2
		Trenton Ch. (d)	1924	1270	200' x13'	0.7	D	1	2400	0.8	15	800	2.5
(12)	HIRAM WALKER DISTILLERY	Old	1900	10	48"	1.2	D	1	12.6	1.2	–	–	–
		New	1955	10	30"	0.8	D	1	19.6	0.8	–	–	–

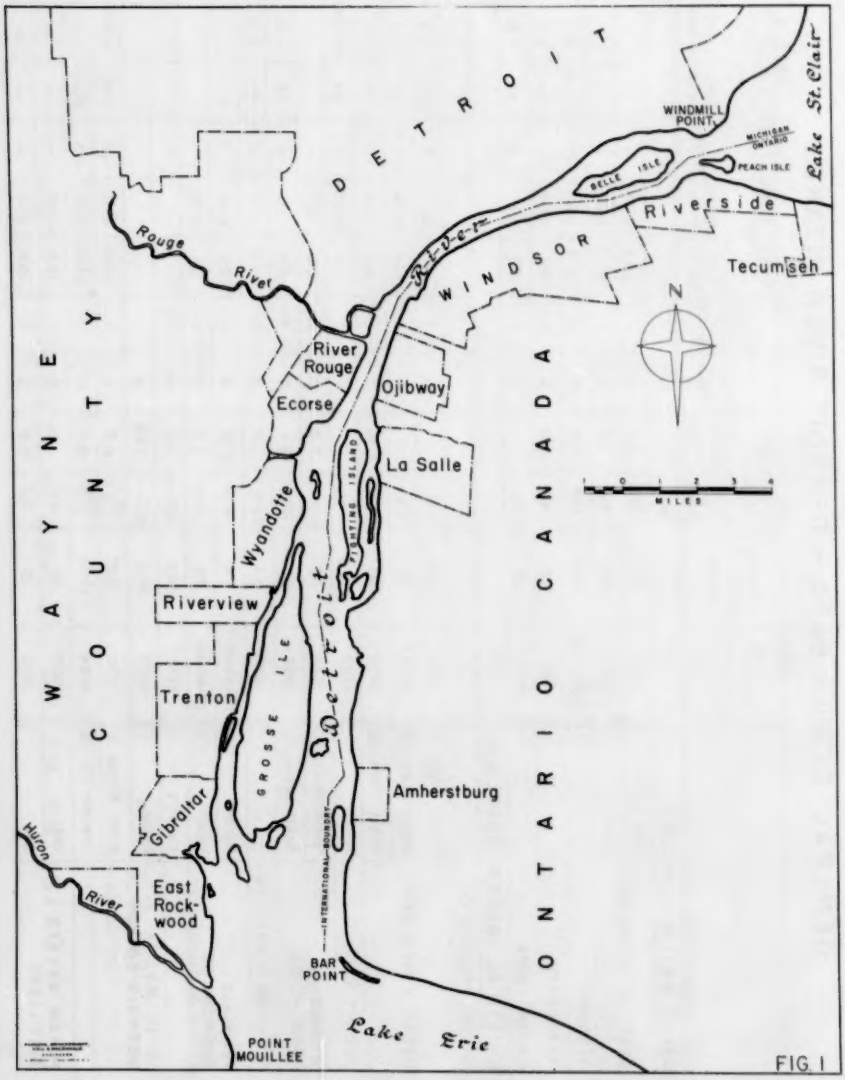

DETROIT
WINDMILL POINT
MICHIGAN ONTARIO
BELLE ISLE
PEACH ISLE
Lake St. Clair
Riverside
Detroit River
WINDSOR
Tecumseh
Rouge River
River Rouge
Ecorse
Ojibway
La Salle
N
0 1 2 3 4
MILES
WAYNE COUNTY
ONTARIO CANADA
Wyandotte
FIGHTING ISLAND
Riverview
Trenton
GROSSE ILE
Detroit
Amherstburg
Gibraltar
Huron River
East Rockwood
INTERNATIONAL BOUNDARY
BAR POINT
POINT MOUILLEE
Lake Erie

FIG. 1

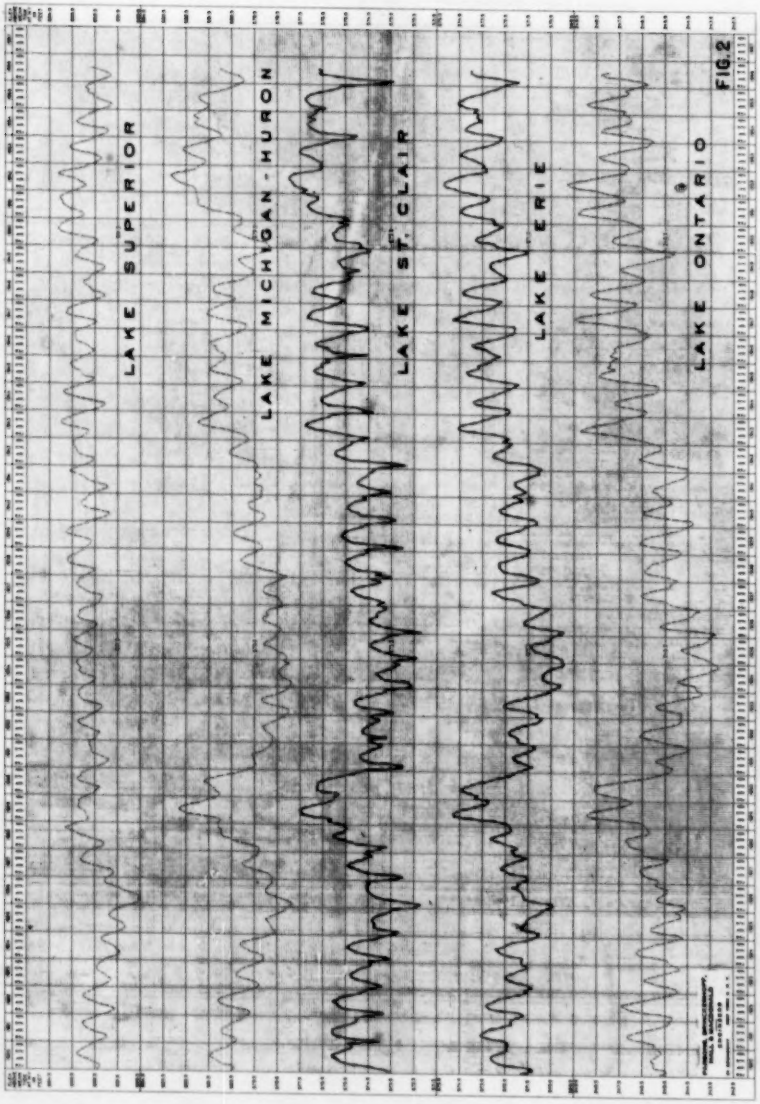

LAKE SUPERIOR
LAKE MICHIGAN-HURON
LAKE ST. CLAIR
LAKE ERIE
LAKE ONTARIO

FIG.2

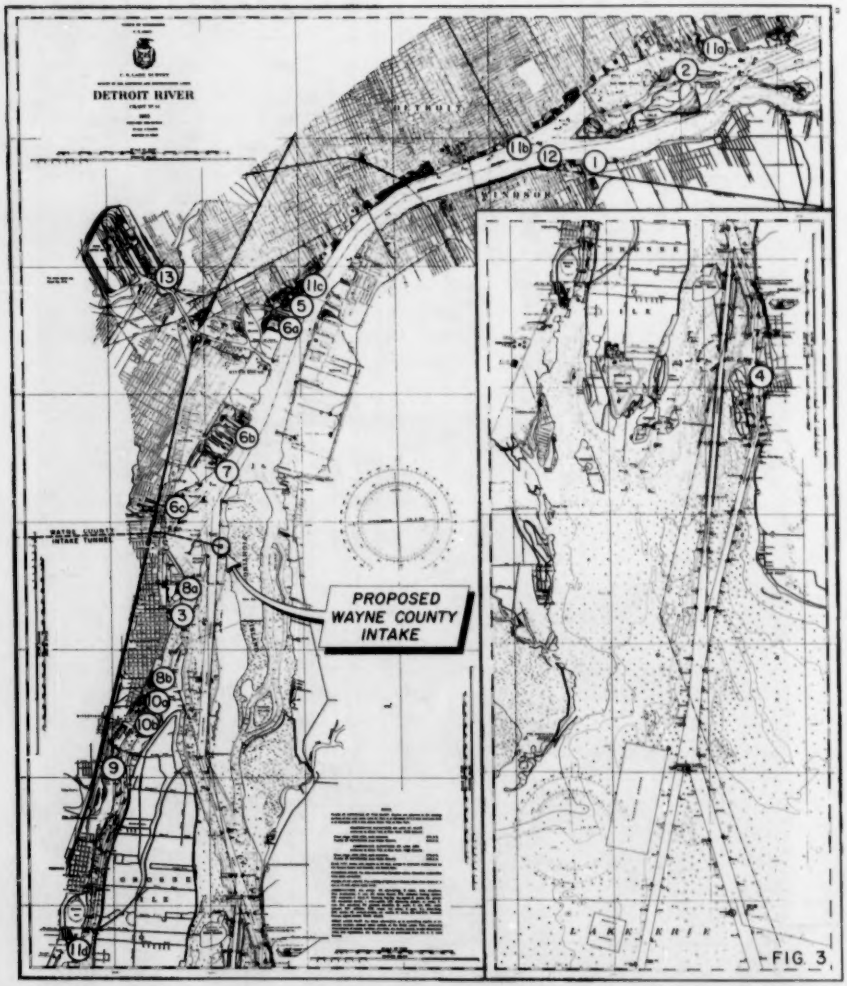
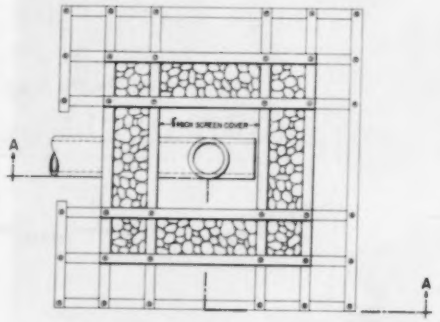
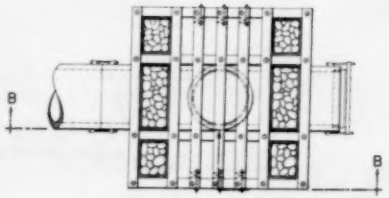


FIG. 3



PLAN



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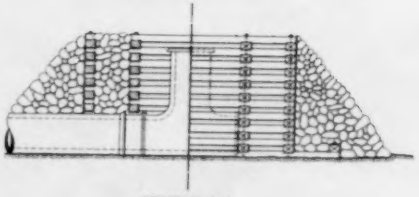
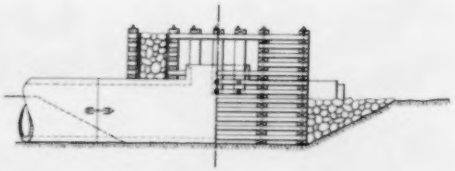


SECTION A-A

OLD CRIB
Built in 1918- Abandoned in 1950

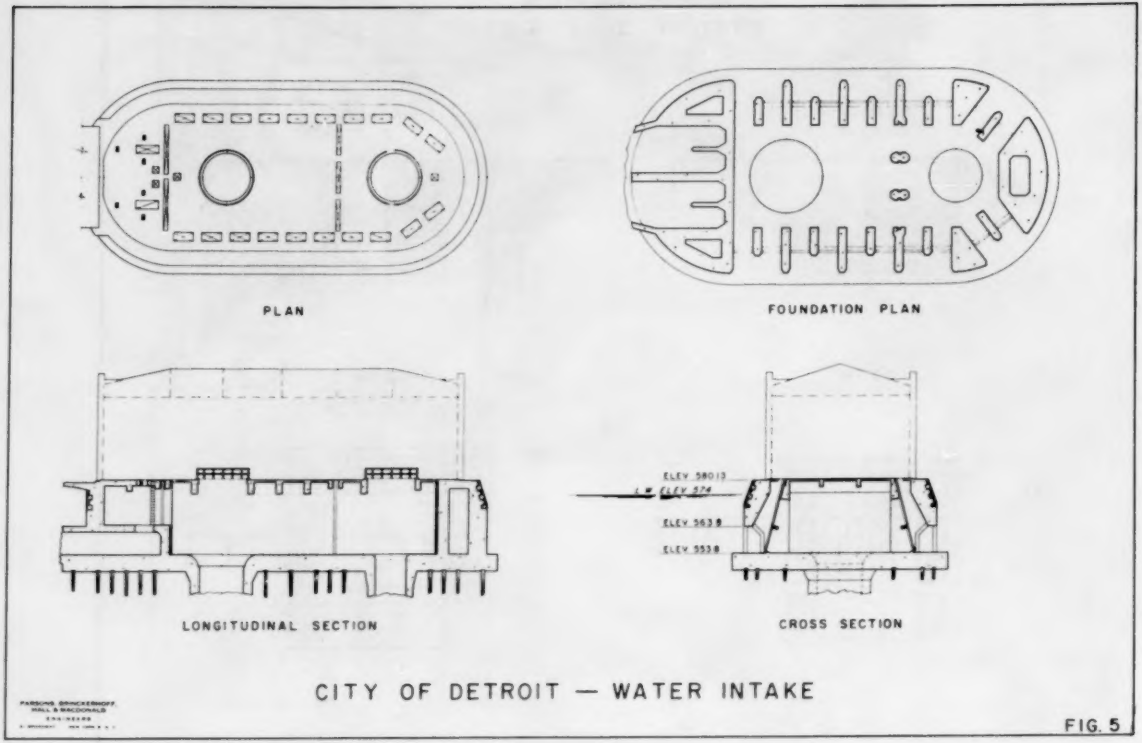


SECTION B-B

NEW CRIB
Built in 1950

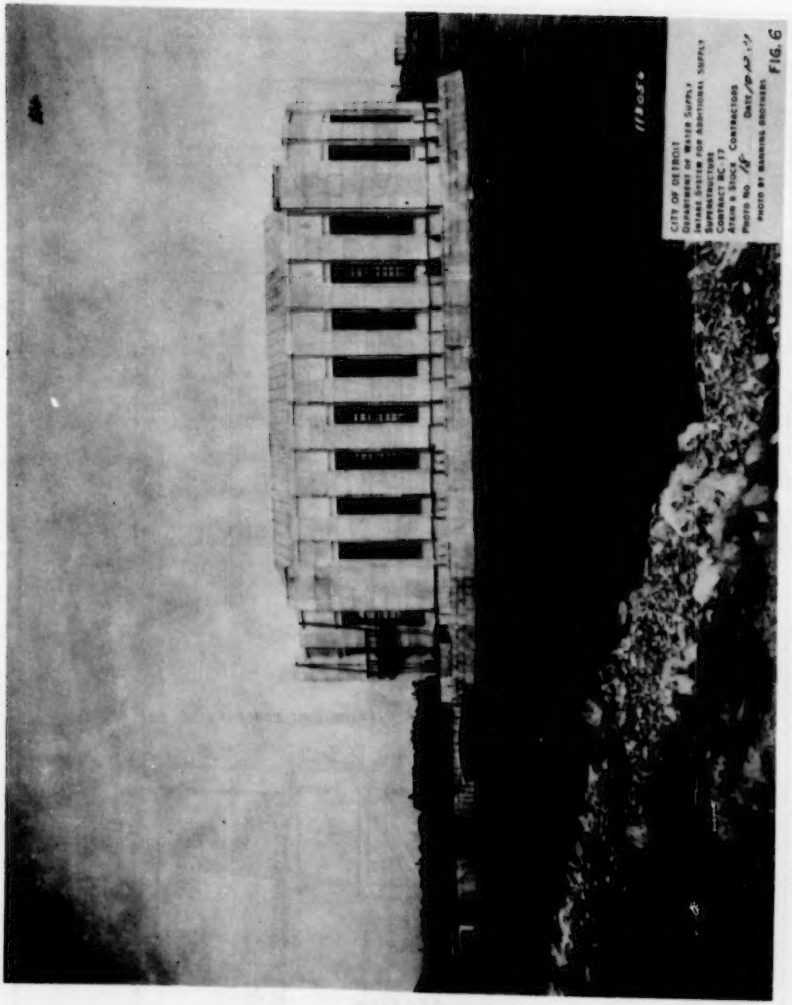
CRIB TYPE INTAKES
WYANDOTTE, MICHIGAN

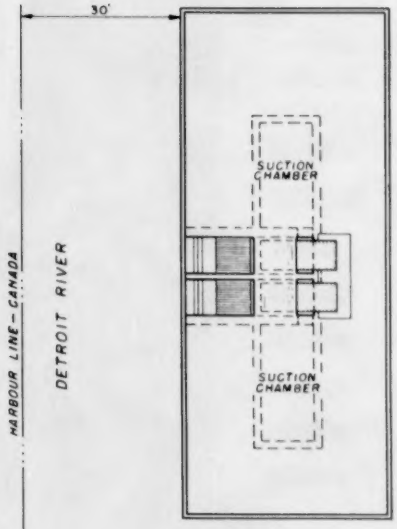
FIG. 4



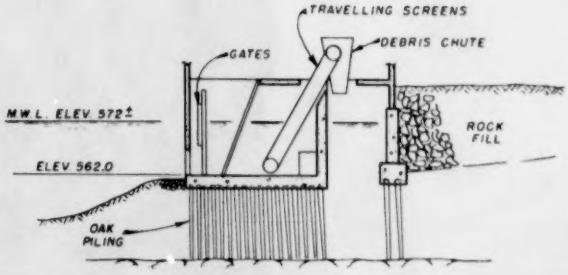
CITY OF DETROIT — WATER INTAKE

FIG. 5


CITY OF DETROIT
DEPARTMENT OF WATER SUPPLY
Intake System for Additional Supply
Lake Huron
Contract WC 17
Photo No. 18
Date 10-26-54
Photo by Manning Brothers
FIG. 6



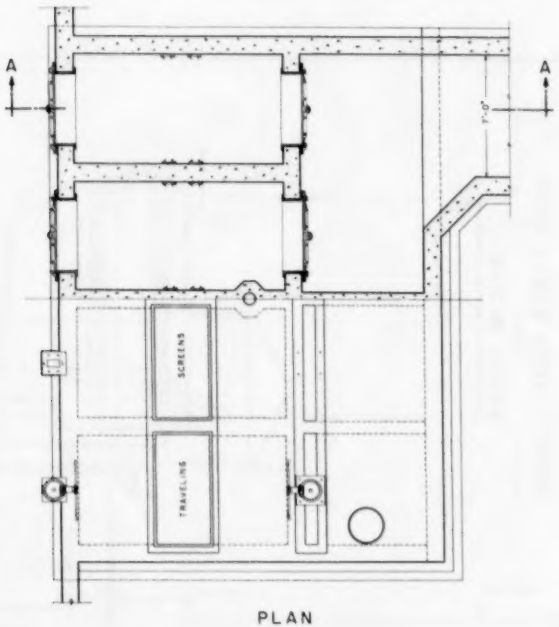
PLAN



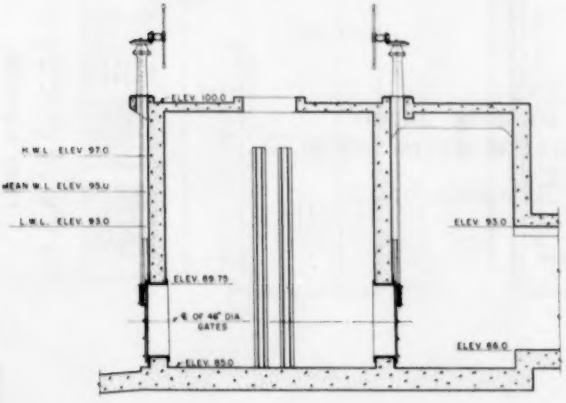
SECTION

AMHERSTBURG, ONTARIO – WATER INTAKE

FIG. 7



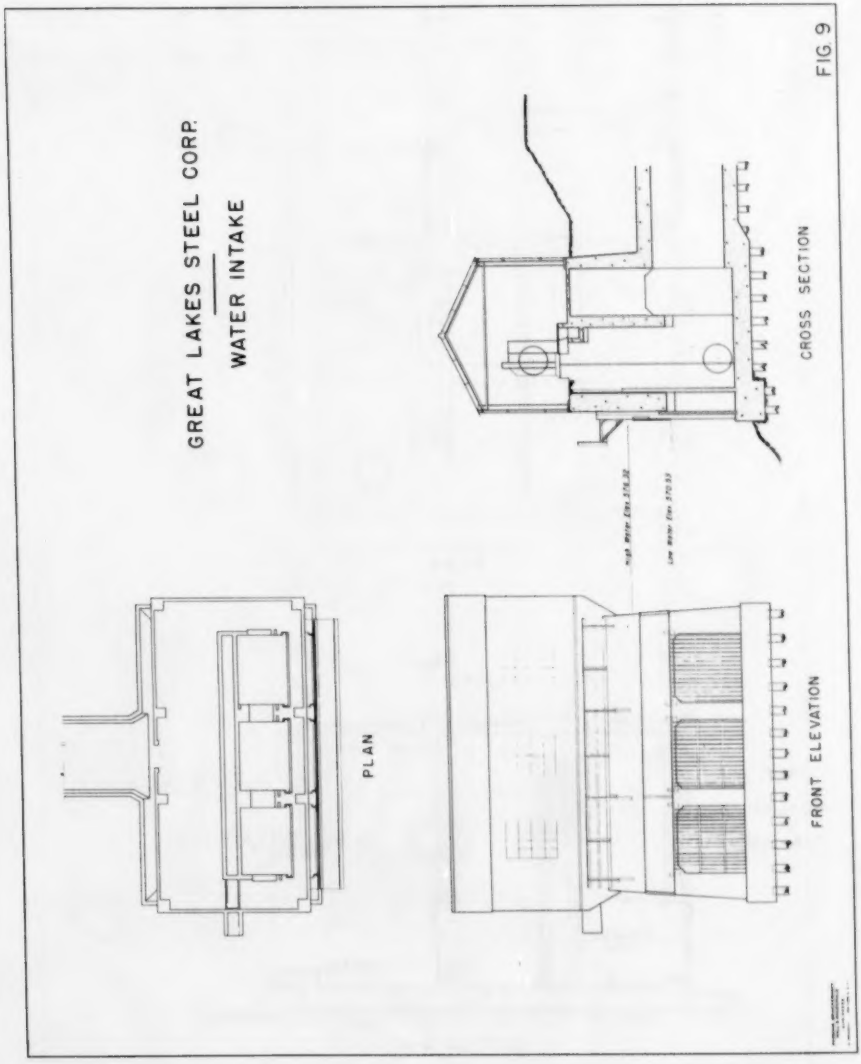
PLAN

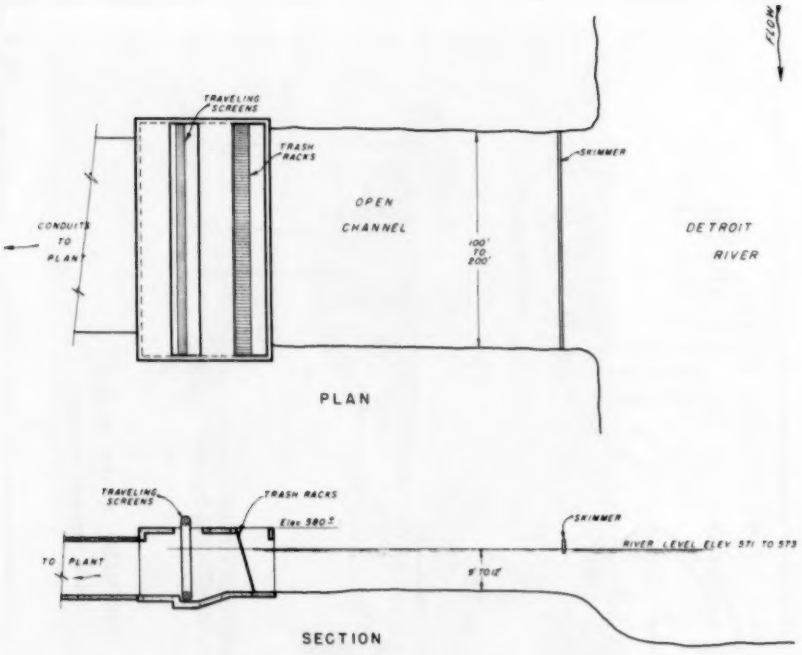


SECTION A-A

PENNSALT CHEMICAL CORP., SOUTH INTAKE

FIG. 8

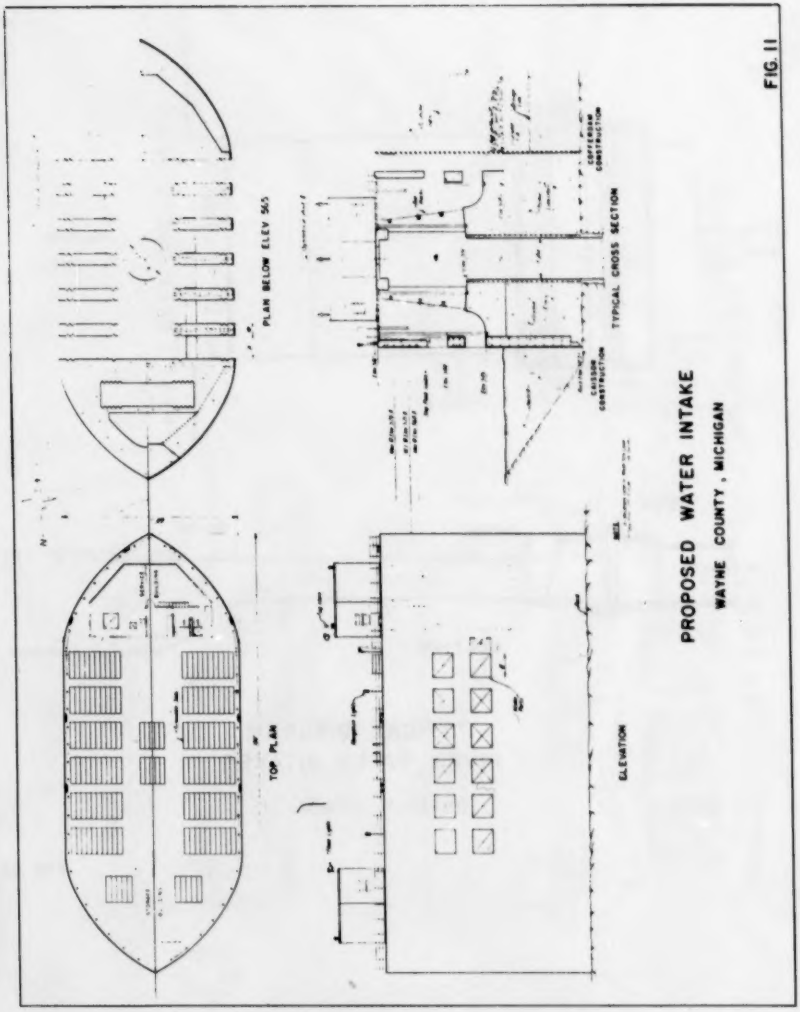

GREAT LAKES STEEL CORP.
WATER INTAKE
CROSS SECTION
FRONT ELEVATION
PLAN
FIG. 9



TYPICAL DIAGRAM
RIVER WATER INTAKES

DETROIT EDISON CO.

FIG. 10


PLAN BELOW ELEV 565
TOP PLAN
ELEVATION
TYPICAL CROSS SECTION
PROPOSED WATER INTAKE
WAYNE COUNTY, MICHIGAN
FIG. 11

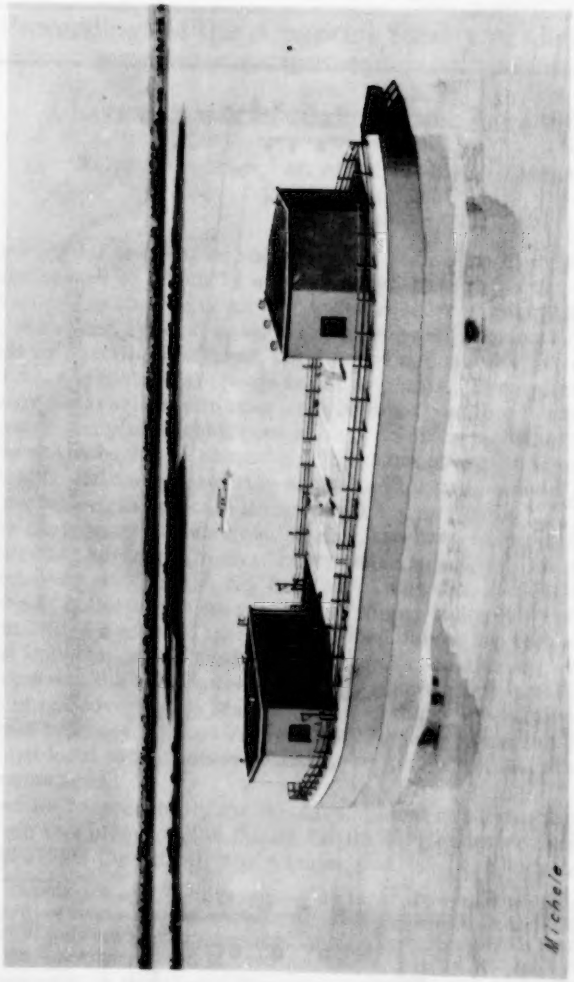
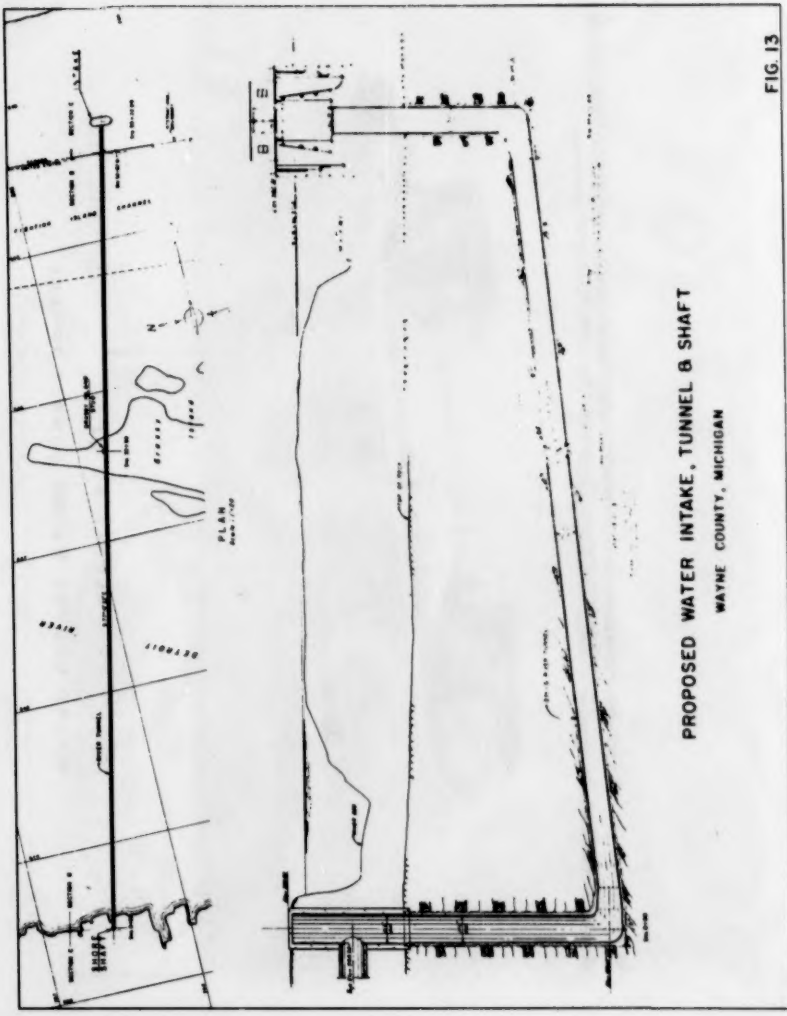


FIG. 12 RENDERING OF WAYNE COUNTY WATER SUPPLY INTAKE


PLAN
PROPOSED WATER INTAKE, TUNNEL & SHAFT
WAYNE COUNTY, MICHIGAN
FIG. 13

Paper 1606 April, 1958 SA 2

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

A NATIONAL WATER QUALITY BASIC DATA PROGRAM[a]

Ralph C. Palange,[1] M. ASCE and Stephen Megregian[2]
(Proc. Paper 1606)

Planning for the best use of our water resources clearly requires continuing intelligence on trends in water quality. Data reflecting such trends are not available in continuity for our water resources generally. The files of local, State, and Federal agencies contain much information covering short periods or specific situations, but a broad national picture of the physical, chemical, and biological trends is not available. Even where long-range surveillance programs are in operation it is not possible to project data nationally because analytical techniques are not uniform, coverage differs with locale, and the analyses do not adequately depict the effects produced by the newer pollutants. Existing state-wide or basin-wide programs in the field are mostly of recent origin and only limited data are available.

The inadequacy of water quality data has been widely recognized. The Presidential Advisory Committee on Water Resources Policy in its report of December 22, 1955, (H.D. No. 315, 84th Congress, 2nd Session) noted the deficiency in collection of data on chemical and biological water quality and recommended a marked increase in expenditures for this activity. In discussing the importance and need for such data in the planning of water resources developments the Committee stated that since the physical distribution of water is not governed by State boundaries there needs to be a primary program supported by the Federal Government. However, it emphasized that State and local participation is extremely important and should be continued and encouraged.

Specific responsibility for the development and conduct of a basic data program was placed in the Public Health Service under Section 4(c) of Public Law 660 (84th Congress), which states that ". . . the Surgeon General shall, in

Note: Discussion open until September 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1606 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 2, April, 1958.

a. Presented at the Annual Convention, ASCE, New York, N. Y., October 16, 1957.

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2. In Charge, Lab. Services, Basic Data Unit, WS &WP Program, San. Eng. Center, Public Health Service, U. S. Dept. of Health, Education, and Welfare, Cincinnati, Ohio.

cooperation with other Federal, State, and local agencies having related responsibilities, collect and disseminate basic data on chemical, physical, and biological water quality insofar as such data or other information relate to water pollution and the prevention and control thereof."

Planning Considerations

In the light of present and anticipated needs the Public Health Service is initiating a long-range basic data collection program and establishing a national network of sampling stations. The program is intended to be a permanent activity, oriented to supplying the knowledge now needed for discerning national trends in water quality. It has evolved from extensive discussions with local, State, interstate, and other Federal agencies. Active local participation is an important part of the operation and will assure development of the maximum amount of information of value locally and nationally. It is expected that the national network will be supplemented with intrastate nets operated by State and other agencies where these are not presently established.

Emerging problems in water supply and water pollution control require that new parameters be evaluated and utilized, such as the measurement of radioactivity and synthetic organic chemical loadings, and the use of aquatic life as sensitive biological indicators of pollution. However, both the old and new parameters must be used, since we must consider all legitimate water uses.

The cost of the program is being shared by the State or local agencies and the Federal Government whenever possible. Local laboratories are being invited to participate by performing the conventional analyses and collecting the samples for the newer analyses. The Public Health Service in turn is responsible for performing the newer examinations and making these results available immediately to the collaborating State and/or local agencies. In addition, there are available to the participating agencies the consultation, training, and other resources which can be provided by the Public Health Service. It is recognized that many laboratories are not now staffed and equipped to carry out the newer analytical techniques. As local resources develop, they can be used to supplement the base PHS network and to carry out the newer analyses.

The program will help to determine needs for new parameters for judging water quality. Development of new analytical techniques will be carried on concurrently as a part of the overall Public Health Service research program. These will be used as developed and needed in the Basic Data program.

Over-All Objectives

The primary purpose of the program is, as indicated, to provide for continuous determination of the quality of the Nation's waters. It will establish base lines for measuring progress in water quality improvement and for detecting early deterioration of our water resources. The principal objectives of the program are, then, to provide:

a. Continuous information on the nature and extent of pollution affecting water quality.
b. Long-term information on changes in water quality as affected by

changes in water use and development.

c. Data which will guide State, interstate, and other agencies in their water pollution control programs.
d. Data on water quality useful in the development of comprehensive water resources programs.
e. Data useful in selection of sites for municipal and industrial water use, and for other legitimate purposes.

Plan of Operations

1. Sampling Locations

Sampling locations were selected to satisfy one or more of the following general categories:

a. Major waters used for public water supply, propagation of fish and wildlife, recreational purposes, and agricultural, industrial, and other legitimate uses.
b. Interstate, coastal, and Great Lakes waters.
c. Waters on which activities of the Federal Government may have an impact.

Final locations of sampling points were fixed after discussions with State and local agencies invited to participate in the program.

2. Laboratory Analyses

Criteria for judging water quality will depend upon the uses which will be made of the waters. Those for domestic purposes should be clear, colorless, taste-free, relatively low in mineral content, and relatively free of deleterious biological and chemical substances. Agricultural water is judged primarily on its mineral content. Water for fish propagation and recreational purposes must be relatively free from domestic and industrial pollution and must be capable of sustaining an active flora of the smaller aquatic organisms on which fish and other wildlife feed. Industrial water quality requirements run the gamut from complete demineralization for certain processes to cooling waters in which temperature is a critical factor. In this program we have endeavored to include those parameters significant to the various legitimate purposes for which the streams are used.

The analyses planned have been selected on the basis of emerging and foreseeable problems in water quality management, as well as those presently existing. They will include:

a. Radioactivity

Interest is rapidly growing in routine monitoring of surface waters for radioactivity. With the development of nuclear power reactors and other peacetime atomic energy uses this interest will become even more intense. State and interstate agencies, as well as municipalities are becoming increasingly concerned about establishing such programs. The Public Health Service has already initiated a reporting system for bringing results from local programs together for compilation and publication on a national basis. Experience with this basic data network will, it is hoped, encourage and stimulate still more effective State and

local programs in radiation monitoring. Programs are already under way within the Service for monitoring the radioactivity of air and the milk supply. This phase of the water quality program will serve to further round out the environmental picture.

b. Synthetic Organic Materials

Wise foresight makes it imperative that background data on the presence of synthetic organics in our surface waters be obtained. This need is made more urgent by the increased use of insecticides, pesticides, fungicides, and other economic poisons, since much of this material reaches watercourses through land drainage. The vast increase in use of detergents in the past 10 years is common knowledge.

Many signs point to the continued rapid expansion of the chemical industries and to their extension to relatively new geographic areas. A doubling of the industry's production capacity by 1975 has been predicted. Expansion of the petro-chemical industry in the interior of the nation has also been predicted.

In some cases even trace concentrations of synthetic organics are sufficient to interfere with domestic water treatment and other legitimate water uses. The standard chemical determinations are not sensitive enough to detect these new pollutants, and very few laboratories are presently staffed and equipped to make this analysis. The CARBON FILTER TECHNIQUE, (encompassing the capture, recovery, and identification of organic substances) is the best tool available for collecting and identifying these materials and is being used in the national network. An added advantage in this analysis is the fact that the material recovered represents a tangible and relatively permanent record of the pollution and remains relatively stable for a long period after collection and isolation. Thus, a more detailed identification can be made later, should the need arise.

c. Biological Populations

Plankton frequently cause taste, odor and treatment difficulties in water supplies. Plankton productivity is directly related to the degree of organic enrichment from fertilizers, human wastes, and industrial wastes of sewage-like character. On the other hand, plankton are important as a food source for fish. Accordingly, a continuous record of plankton populations provides information of importance to water supply problems, to fish productivity, and to water pollution control in general.

Plans are being made for future sampling for bottom organisms and fish, as availability of funds may later permit. Current activities will include assembly of information on sport and commercial fishing, as such may be related to water pollution, from reports and publications on Federal agencies, State Water Pollution Control Agencies, and others on present and proposed programs in both clean and polluted waters.

d. Coliforms

The presence of coliform bacteria in surface waters is generally considered evidence of pollution from human or animal origin. Data on coliforms will help to point up the trends in the effectiveness of control of domestic waste discharges.

e. Conventional Significant Chemical and Physical Parameters

Conventional analyses to be provided by participating agencies will include biochemical oxygen demand (B.O.D.), dissolved oxygen (D.O.), chemical oxygen demand (C.O.D.), chlorine demand, pH, color, turbidity, temperature, ammonia, nitrogen, chlorides, alkalinity (or acidity), and hardness. These will be determined routinely. Selected additional determinations will also be done routinely at locations where special problems exist. The roles of these determinations in evaluating water quality are well recognized.

f. Analytical Reference Service

In collecting analytical data from a variety of sources, measures are needed to assure their precision and accuracy. The Analytical Reference Service has been designed to meet this need and will be utilized by the State and local participating laboratories. It assists them in improving their analyses and data.

The laboratories are provided with synthetic standard samples whose qualitative makeup is given, but whose concentrations are unknown to the participants. The results are returned to our laboratory for comparison with the known quality and with the results of other participants. The results from all participants are then evaluated and reported. In the overall report, individual laboratories are identified by code numbers known only to the Public Health Service. Thus, each laboratory can evaluate its own precision and accuracy on individual analytical procedures against the experience of other laboratories having similar responsibilities. A reliable means for evaluation of old and new analytical methods is also provided.

The Reference Service has met with enthusiastic response from about 40 Federal and State laboratories which have utilized it. Some 50 additional laboratories will be included as a part of the Basic Data Program.

3. Sampling and Laboratory Schedule

The sampling and laboratory schedule in the Basic Data network will be as follows, for each sampling point:

Radioactivity of Water	Weekly
Radioactivity of Plankton	Monthly
Synthetic Organic Chemicals	Monthly
Plankton Populations	Monthly
Coliforms	Weekly
Conventional Physical-Chemical	Weekly

4. Handling and Distribution of Data

The data obtained in this program will be furnished to the participating agencies and to the States concerned on a routine basis as soon as available. Reporting forms for this purpose are now being prepared. All data will be entered currently on machine sorted punch cards for ready compilation. It is anticipated that results for a full year will be ready for printing within one month following the close of the data year.

The Public Health Service plans to issue an annual compilation of all data

in the network, as a Government publication available for general distribution. This compilation will have essentially three parts; (a) a section outlining the philosophy, objectives, and other aspects of the Water Quality Basic Data Program; (b) a section outlining the significance of the various analyses reported; and (c) a section containing the raw data from each station. The data on radioactivity in water, now being assembled from local laboratories not included in the Public Health Service network, may be included as a part of this Government publication.

SUMMARY

With increasing populations and the expansion of industry our pollution sources are growing. The present available information on water quality is inadequate to indicate long-term trends and to otherwise meet existing or anticipated needs in the development of our water resources.

The Public Health Service program for the collection of basic data on water quality will provide information on the chemical, physical, biological, and radiological quality of the Nation's waters on a long-term basis. The network activity is being conducted cooperatively with State, local, and other agencies, and initially will include some 50 sampling stations on interstate and Great Lakes waters.

The program is designed as a permanent but dynamic one. New analyses will be included as their need or usefulness is discerned, while others will be eliminated as circumstances indicate. It is hoped that the ultimate achievement will be an integrated Federal-State-local network. The information developed by such a network is essential to the soundest use of our natural water resources.

Paper 1607 April, 1958 SA 2

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

WATER INTAKES IN THE NIAGARA RIVER AND LAKE ONTARIO[a]

Raymond H. N. Murray[1]
(Proc. Paper 1607)

This paper covers the design of water supply intakes for the Town of Tonawanda, New York, located on the Niagara River and the City of Oswego, New York, located on Lake Ontario. Though the two intakes have many points of similarity, they are different in so many respects that they will be discussed separately. The Town of Tonawanda completed and put into operation its new water plant, including the intake, in May 1956. The City of Oswego's intake and pumping station are now under construction. It is planned to have them in operation by March 1958. Since the Town of Tonawanda's intake was completed first, and some lessons learned in its design were used in the design of the Oswego's intake, it will be discussed first.

Tonawanda Intake

The Town of Tonawanda lies immediately north of and adjacent to Buffalo, New York. The word "Town" here has the same meaning as "Township" in other sections; that is to say, it designates a unit of government administration in a suburban section. The western border of the Town is the Niagara River. The Town has been a rapidly growing suburban section of Buffalo for many years, and has been plagued with water shortages since the start of its rapid growth. It has always owned its own water distribution system, but previous to the completion of the new intake and water treatment plant in May 1956 bought water from the Erie County Water Authority, which took over the service facilities of the Western New York Water Company in 1953. Previous to the formation of the Authority, the Town decided, since it borders on the Niagara River, to install its own water production facilities and started engineering studies to this end. Therefore, when the Authority was formed, the Town requested the State Legislature to exempt it from the service area of the Authority.

The Town, according to the 1950 census, had a population of 55,000 persons. Today the population is over 95,000. It is estimated that the ultimate population of the Town may reach 130,000 persons. On the basis of the 1950

Note: Discussion open until September 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1607 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 2, April, 1958.

a. Presented at ASCE Convention, Buffalo, N. Y., June, 1957.

1. Nussbaumer, Clarke & Velzy, Cons. Engrs., Buffalo, N. Y.

population figures and the potential industrial development, it was determined that the water plant should be designed with a present capacity of 16 million gallons per day, with sufficient land area at the site for future development to 48 million gallons per day. Since there are industries near the plant site that have use for untreated water, it was decided that the intake should have a minimum capacity of 70 million gallons per day. With this additional intake capacity over estimated plant capacity, it would be possible for either the Town or some industry to locate another pumping station adjacent to the intake shaft for supplying untreated water for industrial use.

The waters of Lake Erie are considered of "good" quality for municipal use. But as they enter the Niagara River they are partially polluted on the American side by industries located along the Lake Shore and along tributary streams that discharge into the Niagara River. Also all waste from the City of Buffalo are discharged into the river after receiving only primary treatment at the Buffalo Bird Island Sewage Plant. Due to the swift currents in the Niagara River, this pollution clings fairly close to the American shore, forming a band of polluted water about 1,000 feet in width along the easterly shore of the river. During extreme weather conditions this band of pollution extends farther off shore, and under certain weather conditions is dispersed to some degree for the full width of the river.

The river opposite Buffalo is about 2,000 feet wide. As the river approaches the northerly limits of Buffalo, it widens out to a width of about 6,000 feet, then divided into two channels forming Grand Island. The channel on the east side of the Grand Island is known as the East River or Tonawanda Channel, and the one on the west side as the West River or Chippewa Channel. Above Grand Island and in the widest part of the river is a sand and gravel bar known as Strawberry Island. For practical purposes of designation, the upstream side of Strawberry Island is considered the division of the river into the two channels.

For the past twenty years or more the New York State Department of Health has not considered the East River water of high enough quality to be used as a source of municipal supply and has not permitted any new water supply intakes to be located in the East River. With this limitation, the Town was limited to locating the site of the new water plant within the first mile downstream of Buffalo, though the Town has a total river frontage of about 2-1/4 miles.

An extensive study was made of the river upstream of Grand Island, covering water depth, currents, weed and grass flows and dispersion of pollution from both the American and Canadian shores. At the conclusion of these studies, it was decided to locate the intake in the center of the river about 2,000 feet upstream of the upper end of Strawberry Island, which point is about opposite Vulcan Street, the boundary line between Buffalo and the Town of Tonawanda. This location is in a pool of water with a Low Water Datum depth of 24 feet, and well within the International Boundary Line.

The river frontage, in the section of the river available, was occupied with large industries except for two parcels of land at the foot of Sheridan Drive which is about one mile downstream from Vulcan Street. These two tracts of land, containing about 15 acres, were acquired for the site of the Water Plant. This placed the intake upstream of the plant site and about 5,800 feet distant.

After the plant site and the intake were located, test borings were made along the line of the profile with one boring located at the riser shaft on shore, one at the site of the intake and four spaced at about 1,000 foot intervals

between. The borings were carried down into bed rock to a point about 10 feet below the estimated tunnel profile or about 70 feet into bed rock. The borings showed the overburden in the river to be of sand and gravel. The rock was found to be predominantly limestone, which occurs in horizontal strata ranging from a few inches to 2 or 3 feet in thickness. Gypsum exists in lesser amounts, and strata of either of these main rocks are commonly separated by thin layers of shaly material. Gas was encountered at two of the test borings, but not in any appreciable quantity.

With the profile and test boring data at hand it was not readily apparent whether it would be more economical to use a pipe or a tunnel conduit for conveying the water from the intake to the pumping station on shore. It was decided to design and take bids on both types of conduit. Two pipe sizes, 60-inch and 66-inch diameter, were included in the proposal form. The 60-inch pipe would have a capacity of 70 M.G.D., the 66-inch pipe 88 M.G.D. and the 7 foot diameter tunnel a capacity of 140 M.G.D.

Due to the high cost involved crossing the channel and the tip of Strawberry Island when bids were received the low bid for pipe conduit was about the same as the low bid for the tunnel work. Since the tunnel had more capacity and would not be subject to damage by shipping on the river, it was decided to use tunnel construction.

The tunnel was designed to be an open air tunnel in rock worked from the shore end only. A shaft or riser well was located on shore. The natural ground elevation at this point was 573. Rock was found at elevation 531.5. The invert of the tunnel was set at elevation 473 or 100 feet below ground elevation. From the riser well the tunnel sloped up toward the Intake at the rate of 6 inches per 100 feet, so as to bring all water seepage into the tunnel to the operating shaft where it could be pumped to the surface. The crown of the tunnel at the Intake was designed to be 27 feet below the top surface of the rock. During construction the contractor requested and was granted permission to flatten the slope of the tunnel in order to stay in a more favorable formation of rock. This flatter slope brought the tunnel out at the Intake about 35 feet below the top of the rock instead of the 27 feet as originally planned.

A 72-inch diameter reinforced concrete pipe connects the riser well on shore into the Pumping Station screen well, which is located about 75 feet farther inshore. The invert of the 72-inch pipe is at elevation 552.17 which is 13 feet below low water datum of the river. A 72-inch circular sluice gate was placed on the riser well end of the 72-inch pipe for controlling the flow of water into the pumping station.

The permanent shore shaft or riser well was located about 25 feet back from the shore line of the river. It has a total depth of 105 feet, 65 feet of which is in rock excavation. It extends 5 feet below the invert of the tunnel. The section is rock, starting at a point 5 feet below the top of the rock, has a 5 inch minimum thickness of concrete lining. The portion in earth and in the top 5 feet of rock is 3 ft. thick reinforced concrete. It is capped with a reinforced concrete removable slab, for access if required in the future.

The contractor used the riser well as an access shaft for constructing the tunnel. Steel sheet piling was driven into rock and was used as an outside form for the 3 ft. thick finished concrete shaft. The top 5 feet of rock was found to be full of large seams, which caused extensive leakage from the river into the shaft. A concrete collar was poured at the base of the sheeting and the top of the rock and extensive grouting done to seal off this seepage. The grouting was not successful in sealing off the water, and trouble was

experienced all during construction with this water falling down the shaft. It was not possible to line the shaft until all work in the tunnel was completed, as the 10 ft. finished diameter did not permit sufficient space to accommodate the hoist tower within the shaft. After the shaft was lined with concrete it was possible to seal off the seepage.

The tunnel cross-section is of horseshoe shape. The finished dimensions are as follows: The upper half of the tunnel is of 3 ft. 6 in. diameter. The ribs below the spring line are arcs of 7 ft. diameter with the center of the arc located at the spring line. The invert of the tunnel is a 7 ft. diameter arc with the center of the arc located at the centerline of the crown. The ribs and crown of the tunnel have a minimum concrete thickness of 5 inches. The invert has a minimum thickness of 6 inches. Tunnel muck was not required to be removed before placing the invert concrete. All concrete used in the tunnel contained 7 bags of cement per cubic yard of concrete.

This particular tunnel section was arrived at after consultation with several experienced tunnel contractors, to obtain the smallest and most economical size possible. The minimum rock bore of the tunnel was 7 ft. 10 in. This was considered the minimum width and depth in which a mucking machine could be used. The lower portion of the rubs was curbed in as it is difficult to drill and shoot plumb ribs below the spring line. Since the concrete lining was being used to prevent future spauling of rock and not to retain or keep out water, a minimum concrete lining of 5-inch thickness was used. Also there was considerable saving in not requiring the tunnel muck to be removed before concreting the invert.

The specifications were written on the basis of using supporting steel ribs and lagging in the event the rock was of such character as to require supporting. At the time the plans and specifications were prepared, there was not sufficient information available on roof bolts and mine tie supports to properly specify them. Provisions were made in the specification to permit their use.

Since all workmen and material had to pass through the shore shaft, it was decided, as a matter of safety, to use steel ribs and lagging for the first 200 feet in from the shaft. The balance of the tunnel was supported by means of roof bolts and mine ties. The plates were placed fairly continuously, and no other form of support was employed. The bolts were 1-inch diameter Bethlehem design, varying in lengths of 3, 4 or 5 feet depending on the type of rock being pinned. They were tightened to a torque of 200-foot pounds by means of an air operated impact tool.

Some water seepage was encountered along the line of the tunnel. Grouting was used at several locations to reduce the amount of seepage. No water seams of any volume were encountered. The maximum pumpage for the entire length of tunnel never exceeded 300 g.p.m. The average was about 260 g.p.m.; of this amount about 100 g.p.m. originated in the access shaft.

Some hydrogen-sulphide gas was encountered, as anticipated from the test borings. The gas was not of such concentration that it could not be handled by the fresh air blower supplying air to the workmen in the tunnel. The gas, in the presence of the moisture in the tunnel, was highly corrosive and attacked the air piping, rails, locomotive and all other metal items in the tunnel.

A test boring had been located at the center of the intake structure. The drill hole was encountered within the bore of the tunnel when the rock excavation reached this point. Excavation of the shaft connecting the tunnel into intake structure was started. The shaft was 10 feet in finished diameter with 5-inch minimum thickness concrete lining. The shaft was excavated upward

about 25 feet above the tunnel. A hole was then drilled through into the river above with the drill rod extending up into the river in order to definitely locate the coffer dam for the intake structure over the shaft.

Since all operations in the tunnel were through the one access shaft no concrete work could be started until all excavation within the tunnel was completed. Upon completion of the excavation of the intake shaft from within the tunnel, trimming and clean-up of the tunnel began. As soon as this work was completed, the concrete lining of the tunnel was started. A steel form, 70 feet in length, was used. Concrete from transit mixers was dumped down a pipe at the access shaft which discharged into mine car in the tunnel. It was then transported to point of placement, where it was fed into a pressure concrete machine which forced it into the form in batches under 100 p.s.i. pressure. With the use of 7 bags of cement per cubic yard of concrete, sufficient strength was obtained in the concrete so that the steel form could be loosened and moved ahead within a period of from 16 to 20 hours after completion of the pour.

The design of an intake crib in the Niagara River posed several problems. The location of the intake was in the center of the river about 2000 feet upstream of Strawberry Island in 24 feet of water as measured at low water datum. The velocity of the water at the site was estimated to be from 2 to 3 miles per hour. The test boring at this point showed about 3.5 ft., depth of gravel over bed rock. The 24 foot depth of water extended about 200 feet in all directions from the site. Old maps indicated that Strawberry Island had at one time extended upstream above and to the east of the site, but the island had been dredged away to its present shape and size. During the spring breakup of ice on Lake Erie, huge ice jams occur upstream of Strawberry Island. While making surveys for the location of the intake, it was observed that heavy flows of grass and moss occur at this location. This section of the river is out of the navigable channel as far as freighters are concerned, but is extensively used by fishermen and pleasure craft.

Considering all the problems posed, it was decided to construct a submerged type concrete intake structure that would take in water above the bottom of the river, be of sufficient depth below the surface of the river to permit the largest pleasure boats to pass over at Low Water Datum, would be heavy enough to withstand ice jams, and be so shaped as to by-pass flows of ice, moss, and grass.

The final design consists of a triangular structure of concrete with the apex upstream and the intake port on the downstream side. The triangular structure is located over the tunnel shaft with the centerline of the structure parallel with the flow of the river. The structure, as measured along the line of the triangle, is 22 ft. 0 in. on the intake side and 25 ft. 11-1/2 in. on the other two sides. It has a total depth of 24 feet and is set into bed rock a depth of 6 feet. The intake port, measured at the outside of the structure, is 7 feet in height and about 20 feet in width, with the lower lip 5 feet above the bottom of the river. It is covered with a reinforced concrete hood. An access opening, 6 feet in diameter, is located directly over the tunnel shaft for access into the tunnel. There is a sump at the bottom of the shaft, 5 feet in depth to catch sand in the event sand settles in the shaft. The upstream apex of the triangle was rounded and battered on top in order to by-pass ice in the event a heavy ice jam occurred during low river stage.

The dimensions of the structure are such that an economical coffer dam was built, using a triangular shaped inner core and a circular outer row of

sheet piling, with the space between the two rows of sheeting filled with sand and gravel from the surrounding area. The inside working area was easily dewatered for construction of the structure and completion of the lining of the shaft.

In order to eliminate as much gamble as possible in the contractor's work, and thereby, obtain the most economical price possible for the work, the items of work for which quantities could be determined were bid by the contractors. These items consisted of: concrete lined tunnel; reinforced concrete intake structure; concrete lined intake shaft; reinforced concrete riser well; and concrete lined riser well shaft. Those items of work that were anticipated, but for which no definite quantities could be determined, unit prices were stipulated in the Proposal Form. They consisted of: structural steel, pins, etc., for tunnel support; dry packing in tunnel and shaft; Portland cement for grouting; sand for grouting; grout pipe; drilling holes in rock; and timber. The stipulated unit prices were derived after consultation with several contractors. They were so priced as to give the contractor fair compensation for work performed, but low enough that he would not take advantage of the prices stipulated.

H. F. Darling was the successful bidder on the major items of work in the amount of $989,200. The stipulated items of work amounted to $243,900 making a total cost of $1,233,100 for the intake.

The intake was put into operation in May 1956 and has been in continuous operation since. During this period it has successfully passed a heavy flow of moss which occurred last summer. No trouble was experienced from anchor ice or frazil ice during the winter months when the temperature of the water was down to 32° F., nor was there any trouble experienced this spring when the lake ice passed down the river, though there was an ice jam in the river above Strawberry Island which completely covered the area of the intake.

Oswego Intake

The City of Oswego is located on the south shore of Lake Ontario, about 35 miles north of Syracuse, New York. It has a population of approximately 24,000 persons which has remained about constant over the past 50 years. The Oswego River passes through the center of the City, and is the Lake Ontario outlet of the New York State Barge Canal system. The City has a lake harbor which handles mainly coal, grain and cement. It is believed the St. Lawrence Seaway will materially increase the activity of the Harbor. The Niagara Mohawk Power Corporation's Central New York electric power generating plant is located on the lake shore within the corporate limits of the City. This plant now has a capacity of 400,000 KW. The Power Company plans to add an addition to this plant in the near future.

The City obtains its water from Lake Ontario. The present water plant consists of a lake intake and a Pumping Station. The present intake consists of a 30-inch diameter, bell and spigot, cast iron pipe extending from the pumping station out into the lake, a distance of 8,800 feet. There is a wood timber crib at the end of the pipe line. The crib is in about 80 feet of water. The pipe line was laid out on the lake bottom for its entire length, with the use of wood crib supports where the lake bottom is uneven. Due to the type of support and wave action on the lake, the pipe has given trouble since first installed. A diver's survey of the pipe line in 1951 showed the pipe joints to be

open at numerous points with a 4 ft. gap in the pipe line about 4,000 feet off shore. These breaks are in relatively shallow water and permit sand and turbid water to enter the pipe line when the lake is rough.

The Pump Station is located at the foot of Fifth Avenue, due west and adjacent to the Power Plant. It originally was steam operated. In 1935 the steam pumps were removed and horizontal motor-driven centrifugal pumps were installed. Three of the pumps are located in a pit below lake level, a fourth pump is located at ground level. During a severe northwest storm in 1955 waves from the lake reached the pump station flooding the station and putting the three pumps in the pit out of service for three days. The intake and pump station were installed in 1911 and 1912. They have a maximum capacity of about 11 M.G.D. each. Present peak demands today are about 10 M.G.D. The only treatment given the water is chlorination, which is applied in the suction well at the pumping station.

Faced with the following facts: that extensive repairs were needed on the intake pipe line to make it satisfactory; that the present intake pipe and pumping facilities were not adequate for any industrial expansion in the City; and that the present location of the pumping station was holding up the expansion of the Power Plant, the largest tax paying unit in the City; the City decided in 1954 to have a survey made of the water system to determine what course they should follow in improving the system. A report was submitted to the City in November 1954.

The report recommended that the present intake and pumping station be abandoned, and that new facilities be provided, including a filtration plant. It was recommended that the new intake have a minimum capacity of 50 M.G.D. and preferably a capacity of 100 M.G.D. if the construction cost were not excessive. A new site for the pumping station and proposed filtration plant was recommended west of Sixth Avenue, which permitted expansion of Niagara-Mohawk's Power Plant.

A study of the U. S. Lake Survey Chart indicated that if an intake was to be located in deep enough water not too far distant from shore, and that would be free of pollution from the Oswego River, it would have to be in the general direction of the present intake, as there is shallow water a greater distance off shore on the east side of the Oswego River and prevailing currents and winds carry the pollution of the river to the east.

The U. S. Engineers now require, on the Great Lakes, that any structure near the shipping lanes on the lakes have a minimum water clearance of 40 feet measured from Low Water Datum, and that any structure in navigable water have a minimum clearance of 30 feet. These limitations dictated that the intake crib would have to be located in water from 50 to 60 feet in depth. The chart showed such depth of water to be about 6,200 feet offshore in a northwesterly direction from the present pumping station. There was suitable land west of the present pumping station site for a complete water pumping and treatment plant. This land was owned by the Power Company. Therefore an exchange of land could be made between the City and the Power Company whereby the City would have a good site for its water plant and the Power Company would have the present pumping station site for plant expansion.

In 1955 the City authorized the preparation of plans for a new lake intake and new pumping station. Test borings, both on shore and in the lake, were also authorized. The borings were made in the summer of 1955.

Trouble experienced with the existing intake pipe indicated that the bottom of the lake was either rock or boulders or both, and that it would be difficult

and expensive to install a new intake pipe conduit. To prevent movement of the pipe, it would be necessary to lay it in a trench for the whole distance. Considering weather conditions on the lake during the average year, it would not be feasible to complete such a pipe line under two years. In view of these problems, a rock tunnel conduit was considered the better, if test borings indicated a satisfactory rock in which to make the bore.

Good borings would be required in the lake to obtain proper data for evaluating tunnel work. At Tunawanda it was observed that waves materially affected the operation of the test boring drill, causing it to chop up and down and thereby break and pulverize the rock core sample. Since conditions on Lake Ontario, not only from waves but from swells, would be much more severe, it was decided to specify that the drill rig be so mounted in the lake that it would not be affected by the action of the water. It was also specified that careful observation was to be made for gas during the drilling; that after each hole was drilled, a water pressure and leakage test be made to determine if the rock was seamy or porous; and that upon the completion of each hole it was to be sealed with cement.

Seven holes were drilled on the line of the proposed intake, one at proposed shaft on shore, one at the intake crib site and five spaced approximately 1,000 feet apart between these points. Empire Soils Investigations, Inc., of Ithaca, New York, made the borings. The drill rig was mounted on a platform supported by a three-legged tripod arrangement. The legs of the tripod were made of two different diameter pipe so that they could be telescoped to accommodate the varying depth of water at each hole. The platform was set about 15 feet above lake level. The deepest water was at the site of the intake crib, which was 60 feet in depth. This hole was drilled first. Then the rig was moved in shore progressively hole by hole. A large lake barge equipped with a crane handled the drilling platform.

Excellent core samples were obtained. In many instances 100 per cent recovery was made. The rock was a dense gray sandstone with occasional thin streaks of shale. There was no appreciable loss of water where pressure tests were made. No gas was observed. The seven borings for the intake cost about $45,000.00.

The test borings showed rock to be approximately 22 feet below ground surface at the shore boring, which placed the rock at elevation 244, Low Water Datum for Lake Ontario. The surface of bed rock sloped down from this point to a point about 2,500 feet off shore, where it was found to be 35 feet below lake bottom. Then it sloped up again to a point about 4,000 feet off shore where bedrock was only a few feet below lake bottom. This depth existed out to the intake crib site where only 18 inches of fine dense sand was found on top bedrock. The dip in the bedrock was filled with boulders and gravel. In fact the bottom of the lake had a rather even slope from shore out to the intake site. There was sufficient unevenness in the bottom to indicate that rock trenching would be required for a large percentage of the way if a pipe conduit was to be installed.

The same problem of whether to use a pipe conduit or tunnel conduit was posed at Oswego as at Tonawanda. It was decided to design and take bids on both type of conduit. A 66-inch diameter reinforced subaqueous pipe conduit was bid as an alternate. It had a capacity of 70 M.G.D. The tunnel as designed has a capacity of 125 M.G.D. The pipe was required to be laid in a trench for the full length. The intake crib for the pipe was of the same design as for the tunnel. When bids were received, the tunnel construction was within $25,000

of the lowest price submitted for the pipe. Considering contingent items of cost, the tunnel was considered to be the cheaper.

The tunnel was designed to be an open air tunnel with the riser well or access shaft located about 25 feet in from the shore line. The cross-section dimensions of the tunnel are the same as used at Tonawanda, except that the concrete lining was made 6 inches in thickness. This greater thickness was used, as it was found in the Tonawanda tunnel the 5-inch minimum thickness was not sufficient to accommodate the concrete fill pipe. The test boring data indicated that the rock through which the tunnel was to be bored was of such density and quality that it might be possible to eliminate the concrete lining. The specifications permitted and the proposal form was set up so that the item of concrete lining could be omitted if it was found feasible to do so. The unlined tunnel would have a minimum dimension of 8 feet, and would have a theoretical capacity with this minimum dimension of 120 M.G.D.

Roof bolts and mine tie plates were specified for roof support where required. Rib type steel supports were also specified, in the event bad rock conditions should be encountered. Roof bolts and mine tie plates have been used throughout the tunnel. The tie plates are spaced at about 6 ft. intervals along the tunnel. Experience in this tunnel indicates, that better support at a more economical price can be obtained by the general use of 8 inch by 8 inch steel plate washers in place of mine tie plates in small bore tunnels such as these two.

The riser well or access shaft to the tunnel is located about 25 feet in from the original shore line. The shore of the lake at this point erodes badly. To prevent future erosion, it is required that all rock from the tunnel and pumping station excavations be placed along the shore line. This fill material will extend out from the original shore line about 170 feet.

The rock outcrop at the shore line is at elevation 244, or at Low Water Datum for Lake Ontario. The elevation of the land was 266. The area was graded off to elevation 261, which left an overburden depth of 17 feet at the riser well. A 72-inch diameter pipe connects from the riser well shaft into the pumping station 75 feet distant. The invert of this pipe is at elevation 232, or 12 feet below the top of bedrock. This pipe line is under the tunnel contract and is to be installed in a rock bore tunnel.

The portion of the riser well shaft from 5 feet below the invert of the 72-inch pipe to the top of the shaft at elevation 261 was made 12 feet square with 3 ft. thick reinforced concrete walls. Two 72-inch diameter sluice gates are located in the shaft. One is to control the flow of water into the pumping station and the other is to control the flow into a future pumping station if some outside community or if industry would require raw water.

Below the 12 foot square section, the shaft is 12 feet finished diameter, with a minimum concrete lining thickness of 6 inches. The 12-foot diameter shaft was used so that there would be sufficient room to accommodate the hoist tower in the event it was found desirable to concrete line the shaft before excavating the tunnel. Seepage at the top of the rock and the encountering of a wet fault in the shaft that flowed salt water dictated that the shaft be concrete lined before starting the tunnel excavation. The experience of working in the lined shaft over working in the unlined shaft at Tonawanda would indicate all access shafts be lined before starting tunnel excavation if any water seepage is anticipated.

The shaft has a total depth of 135 feet. The invert of the shaft is 5 feet below the invert of the tunnel at 131. This 5 foot sump acts as a pit for dumping

muck cars during tunnel excavation and will act as a sediment sump in the future.

The tunnel slopes up from elevation 131 at the riser well shaft at the rate of 3 inches per 100 feet for the full 6,200 feet, and terminates at intake shaft at elevation 146.5. This slope permits all tunnel seepage and drill water to run to the foot of the tunnel where it is pumped to the surface.

The top of the rock at the intake end of the tunnel is at elevation 188.5 which gives an intake shaft depth of 42 feet. This shaft is to be 10 feet finished diameter, with a minimum lining thickness of 6 inches. A 12 foot diameter steel pipe is to be inserted in the top of the shaft. The pipe is to extend down from the top of the rock 10 feet, and 5 feet above the bottom of the lake. There is an 18 inch layer of sand on the bottom of the lake, making a total pipe length of 16 feet, 6 inches. The pipe is to be set in concrete.

It is the intent to break through from the lake into the tunnel and to place the intake crib without the use of a coffer dam. Both the intake shaft and the crib were designed with this proposal in mind. The decision to attempt this type of construction was based on the fact that the work is in open water 60 feet in depth. A coffer dam of adequate dimensions at this location would be very costly.

When the tunnel reaches the intake shaft it is intended to stope up in the shaft as far as the contractor thinks safe. A pilot hole will then be drilled through into the lake to determine the quality and the water tightness of the rock. After the excavation has reached a maximum height considered safe, the tunnel will then be 100 per cent completed and flooded with water, after which the rock plug left will be excavated with marine equipment from the lake. The last shot of rock will be permitted to drop into the bottom of the shaft. The contractor is permitted to excavate a sump at the bottom of the shaft of sufficient depth to receive this rock. All spoil from the shot is to be removed from the shaft to a point at least level with the invert of the tunnel.

The crib is designed of steel. It is octagonal in shape and measures 54 feet out to out. The outer periphery of the structure consists of 8 steel compartments measuring 7 feet in width by 14 feet in depth, which are to be filled with concrete after the structure is in place. Steel trusses connect the outer compartments at their intersections. The trusses connect into a steel drum at the center. A 20 foot octagonal shape steel hood, with 5 foot depth side baffle plates, covers the pipe opening into the tunnel shaft. The space between the hood and the periphery chambers consists of a wood grill for entrance of water into crib. The grill is made up of 3 inch by 12 inch timber bolted to the top of the steel trusses joining the outer chamber. The structure is designed of such dimensions that the passage of water through the grill at the rate of 125 M.G.D. will be 4 inches per second, between the baffle and the 12 ft. shaft pipe 7.5 inches per second, and in the 12 ft. shaft pipe 20.5 inches per second. The minimum thickness of steel plate used is 3/4 inch and structural members 5/8 inch. All surfaces of the structure except the baffle plate and the top of the shaft pipe are to be coated with bitumastic enamel. The baffle plate and the top of the pipe are to be rubber coated to prevent water from freezing to these surfaces.

The area occupied by the structure is to be graded off to a level surface and covered with an 18 inch thickness of No. 4 broken stone. The outer compartments are designed to be airtight so the structure can be fabricated on shore and floated into place. When the structure is in position the compartments can be flooded as desired to lower the structure into place.

After the crib is in its final place, the compartments are to be filled with concrete, and the whole outer periphery banked with 5-ton boulders for the full height of the structure and sloping out from the structure at an angle of 45 degrees.

H. F. Darling, Williamsville, New York, have a $1,753,415 contract covering the intake, including the crib. They have sublet the marine work and crib to Merritt-Chapman Scott Corporation, Cleveland, Ohio. Work on the project was started in April 1956. The tunnel excavation is about completed and will be ready for the marine work to start in July 1957. The rock encountered has been of excellent quality and it is not proposed to line the tunnel. Several small vertical faults in the rock have been encountered. Several of these have seeped salt water. It has been possible to seal off this seepage by means of grouting. Seepage into the 6,200 feet of tunnel does not exceed 10 gpm.

Construction of the pumping station started in April 1957. The total contract price is $1,055,151. McElwee-Courbis Construction Company, Camden, New Jersey, have the general contract. George Clucas, Inc., Buffalo, New York, have the mechanical work. Buffalo Electric Company, Buffalo, New York, have the electrical work.

Nussbaumer, Clarke and Velzy, Consulting Engineers, prepared the original report, and the construction plans and specifications for both projects and are supervising the construction of the work.

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DIGITAL COMPUTERS FOR PIPELINE NETWORK ANALYSIS[a]

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(Proc. Paper 1608)

ABSTRACT

Methods of solving engineering problems are always undergoing revision and improvement to reduce the tedium of repeated calculation, to reduce the overall costs and in many cases to improve the accuracy of the results. Digital and analog computers are an important portion of the equipment available for reducing the time required to solve many of these problems. This paper on digital computers for the solution of pipeline network problems sets forth one of the recent developments which should provide added economy in engineering design.

INTRODUCTION

The solution of the problems in pipeline networks with the aid of a digital computer is in itself a worthwhile undertaking. In addition, the knowledge of digital computers gained in this particular application should prove of great value in the solution of other engineering problems.

One of the most dramatic and significant developments of our time is the electronic or digital computer. Arithmetic computations in near limitless quantities by means of accurate, inexpensive methods enable us to expand our knowledge on almost every front. Research in the fields of nuclear energy, various branches of engineering, commerce, weather, and government is being accelerated by the solution of problems, many of them involving billions of arithmetic operations. Several thousand electronic calculators are now in

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operation.[18] They are solving problems ranging all the way from the calculation of the simple payroll to the solution of the most complicated problems in such fields as nuclear physics and hydrodynamics. The full potentialities of the electronic technique for computations are only beginning to unfold.

Some idea of the productive speed for computers can be obtained by considering the multiplication of two large numbers, say 10-digit numbers, to form their product. If one were to form a thousand such products, the time required by various methods would be roughly as follows:[23]

By hand with pencil and paper	1 week
With the aid of a key-controlled mechanical desk calculator	1 day
By electro-magnetic calculator with automatic reading and writing	1 hour
By small electronic calculator with automatic reading and writing	1 minute
By electronic supercalculator (NORC, etc.)	1 second

At the rate of one thousand multiplications per minute, the output of an eight-hour day is nearly four million operations. Many of the larger calculators are operated around the clock with little idle time. Some machines have a tremendous backlog of work piled up and will be replaced by more efficient and faster units as the need and economics of operation permits.

There is a tendency for some engineers who have been in practice varying lengths of time to view such machines with considerable skepticism in view of the complexity of the machines. However, not only have fast and economical circuits been devised but the machines have been arranged to facilitate their use. Special systems for programing various problems have been devised which aid the engineer in the solution of many types of problems. The IBM 650 digital computer at the Computer Center of Oklahoma State University is relatively easy to understand and operate. Nevertheless, the learning process required to become efficient in the use of computers requires time and practice. To learn what this machine does and to learn a good deal about how it works is comparatively easy. With a fairly small amount of time spent in studying the machine and programming techniques one may learn to solve relatively simple problems. To obtain an adequate working knowledge of the machine and its possible applications, including programming techniques requires interest, time and effort.

There are many different interpretive methods for programming a problem on the digital computer. The manufacturer of the machines provides the users with all available programming techniques, literature, and general machine use information available. Choice of the particular method to use in solving a problem is governed by the extent of one's knowledge of the several methods available. It may be possible to solve a particular problem in many ways using the same programming technique or interpretive procedures with modifications. Considerable study is required to determine the most convenient and most efficient method to use for a particular problem or application.

While the exclusive use of a digital computer is limited to those firms and institutions which have large volumes of highly repetitive computations to perform, it is possible for those with a more limited volume of work to benefit from the use of machines installed in institutions, at manufacturer's

computing centers and at commercial calculating offices. The greatest savings in time and manpower brought about by the use of a digital computer is on those problems which require many repetitive computations. Where the computers are in use and program and machine time are available it could prove profitable to solve less repetitive problems for practice and machine efficiency. This would be especially true where a company rents the equipment and would be required to pay the rental whether it is in use or not.

In the fields of civil and sanitary engineering problems involving stress analysis, surveying (traverse adjustments, earthwork computations), traffic flow analysis, and sanitary and water system problems offer many opportunities for computer application. It must be kept in mind that, where the amount of computation is small, types of computers other than the electronic digital computer will usually prove to be the most economical and satisfactory.

There are applications for which the analog type of computer will prove to be more desirable than the digital type. Following a description of the equipment and procedure used in an analysis of a water distribution system some of the advantages and disadvantages of digital and analog (McIlroy) computers will be discussed as far as they apply to water distribution systems.

Programming for the IBM 650 and Similar Computers

In applying the IBM 650 to the solution of a water distribution system network the Hazen-Williams formula for flow in pipes was used in computing the relation between the head loss and the discharge. A modification of the method of water distribution system analysis as developed by Professor Hardy Cross was programmed for the digital computer. The actual example cited is for the solution of a simple two circuit water distribution network. While such a network may be more conveniently solved by means other than the digital or analog computers it is used here for ease of explanation. It is evident that there are many methods for the solution of distribution system problems with electronic computers. The program technique used here in the included example should lend itself to the solution of complicated hydraulic circuits or networks with some adaptation of the program to the particular problem.

Before illustrating the solution of a network with the digital computer it is necessary that certain basic fundamentals of computer operation be well understood. The "650" computer, manufactured by the International Business Machines Corporation, is a moderate speed moderate capacity machine.(15) An addition takes place in about five-thousandths of a second, and a multiplication in about twelve-thousandths of a second. Because of these comparatively high speeds, extensive calculations can be done in a reasonable length of time.

The "memory" of the machine is a magnetic drum capable of retaining two thousand ten-digit numbers or "words" with their associated signs. Data upon which the machine works and instructions governing the machine operation are all in the form of such ten digit numbers and are all stored on the magnetic drum. Instructions, therefore, are indistinguishable from other data and at the discretion of the programmer they can be modified by the common arithmetic operations. The fact that instructions are stored within the computer leads to the term "stored program computer". Once data or instructions are stored on the magnetic drum they will remain there until altered by the program or until they are erased.

Information, both data and instructions, is introduced into the computer through punched cards. Results are obtained from the machine in the same form. To obtain a printed record, the cards punched by the 650 are run through a tabulating machine. The general procedure for using the 650 computer will make use of the following steps:

1. Programming: After the mathematical procedure to get the desired results is well understood, a program of instructions is written together with the data.
2. Keypunch: The program and a sample set of data are punched on cards. The program should be punched with one instruction per card for the first run.
3. Verify: The punched cards must be checked for punching errors.
4. Debug (or check out) the program: For this part of the process it is essential that the programmer has worked out in advance a numerical case. Use a simple set of data and record the intermediate results.
5. Consolidate the program onto a working deck of punch cards for future work. Usually either six or seven instructions can be placed on each card.
6. Duplicate the working deck. This is done mechanically on the IBM reproducer.

When these six steps have been successfully completed, the program is ready for production use. Once the program is worked out for a particular network it may be adapted rather easily to other networks.

A program is a set of instructions telling the machine what operations to perform on a given set of data. Each instruction must contain an operation code and the address or location in the memory of the number or word upon which the operation is to be performed. Each instruction contains ten digits. The form in which the instruction is written, that is, which digit positions refer to the operation code, and which to the data address may vary depending on the system in which the program is written. The various ways of writing instructions are referred to as "languages." The basic form of instruction is called "machine language" or "native language." The machine can be made to execute other types of instructions. A program, stored on the drum treats the new type of instruction, sometimes called pseudo-instruction, as its data. This program analyses the pseudo-instructions, and interprets, in terms of the machine's native language what the pseudo-instructions mean. The purpose of devising auxiliary languages, or "interpretive systems" as they are frequently called is to provide an improved communication between the machine and the programmer for certain types of problems. Some of the immediate advantages that should be achieved by using such systems are:

1. They should simplify program writing.
2. They should make each instruction more effective.
3. They should ease the task of testing the program.

The machine language continues to be available even when the interpretive system is being used.

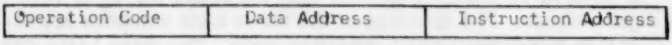
It is possible to devise many types of interpretive systems and indeed, various types have been set up. One of these, the IBM Laboratories Interpretive System called "Flops" has proved particularly successful and is used at the Oklahoma State University Computer Center. Before discussing the Flops interpretive system the machine language type of instruction will be considered

briefly, as some ideas are common to both.

An instruction is always a 10 digit word. Data which are to be operated on are also entered as 10 digit words. While the programmer knows whether a word entered into the machine is data or instruction, no actual differentiation is made on the drum of the machine.

As previously stated the "memory" of the 650 computer is a magnetic drum on which 2000 ten digit words may be stored. The storage location of each word is designated by a number, 0000 through 1999. This number is called the "address." It is important to keep in mind the difference between the addresses of the data or instructions (4 digits) and the data or instructions themselves (10 digits).

The machine language instruction is a ten digit word containing a two digit operation code, a four digit address of the data to be operated on and a four digit address of the next instruction which is to be performed. This is illustrated by the following diagram:



It is possible to have 100 (00 through 99) operation codes. Only 44 have been assigned a meaning in the basic machine. This set of 44 operations was selected by the manufacturer to contain those which were expected to be basic to the solution of most problems and convenient from the standpoint of economic and engineering considerations. Since each instruction contains the location of the next instruction it is not necessary for the instructions to be performed in the order of their addresses. Nevertheless, when a programmer writes a program care must be used to see that the series of instructions form a chain, until finally there is an instruction to stop the program. After the program has been constructed any addresses not used for instructions are available for storing the data which are to be processed.

When data are entered into the memory to be processed in machine language, the decimal point is assumed to be located in front of the ten digits. That is, all words are assumed to be less than one in magnitude and the programmer must keep track of where the decimal point of his answer should be. Also, the programmer must enter the data in such a way or issue proper instructions so that the words which result from any arithmetic operation will never exceed 0.99 9999 9999. For a complete description of machine language codes and programming the operation manual for the particular machine to be used should be consulted.

FLOPS is an interpretive system in which the machine language instructions making up the system fill addresses 1000-1999 on the drum. Only the addresses 0000-0999 are available to the user for data and instructions. All of these addresses can be identified by making use of the last three digits, only. Therefore, a FLOPS instruction can make use of three digit addresses, the leading zero being understood. The address 0000 (or 000) is reserved for a special use. It is called the previous result register.

FLOPS differs from machine language in three basic ways:

1. In the form of the instructions
2. In the meaning of the operation codes and
3. In the way in which the location of the decimal point of a given word is expressed.

The usual form of a FLOPS instruction does not contain the address of the

next instruction. Therefore, it has been arranged that instructions be taken sequentially until the sequence is changed by one of a number of "transfer" instructions. The programmer must be careful that the instructions form a chain containing only instructions although most of the chain will be in sequence.

There are five parts to a FLOPS instruction. The first part is the sign. Most instructions will have a plus sign associated with them, but the minus sign will be used under certain conditions.

Sign	O_1	A or O_2	B	C

The operation code may be either a one digit code O_1 with values of one through nine or if O_1 is zero the operation code will be a three digit code O_2. The operation codes that have been assigned are listed in Table 1. The details of how the operations are carried out are not shown.

When O_1 is used as an instruction code then the space for O_2 is used as an "A" three digit address. The meaning of the three digit numbers appearing in A, B, and C varies depending on the operation code. The third basic difference between FLOPS and machine language, as pointed out previously is the way in which the location of the decimal point of a given word is expressed. The FLOPS system utilizes what is known as the "Floating-Point" system of determining the decimal point. This system is particularly useful in many engineering applications.

Scientists and engineers whose problems involve numbers with a wide range of values use this "floating-point" system on the digital computers. In this system, every number stored and operated on is considered to be in a predetermined "standard form" consisting of a "normalized" fractional part and an accompanying power of the base. The normalized fractional part is usually a fixed point number, with radix point at the left while the "exponential" is treated as an integer by the machine, with the radix point at the right. In addition, normalized numbers are considered to have their first non-zero digit just to the right of the radix point. This guarantees the carrying of as many significant digits as possible. After each arithmetical operation is performed, the result is again placed in the standard form. Floating-point operation thus provides for automatic scaling, automatic prevention of overflow, and automatic guarantee of as many significant digits as possible in the result for the given arithmetic sequence.

Computer Application to Flow Network

The example shown by the Flow Diagram (Fig. 1) was solved on the IBM 650 digital computer using the FLOPS interpretive system. Storage spaces on the magnetic drum were assigned to the instruction program and other storage spaces on the drum were assigned to the data.

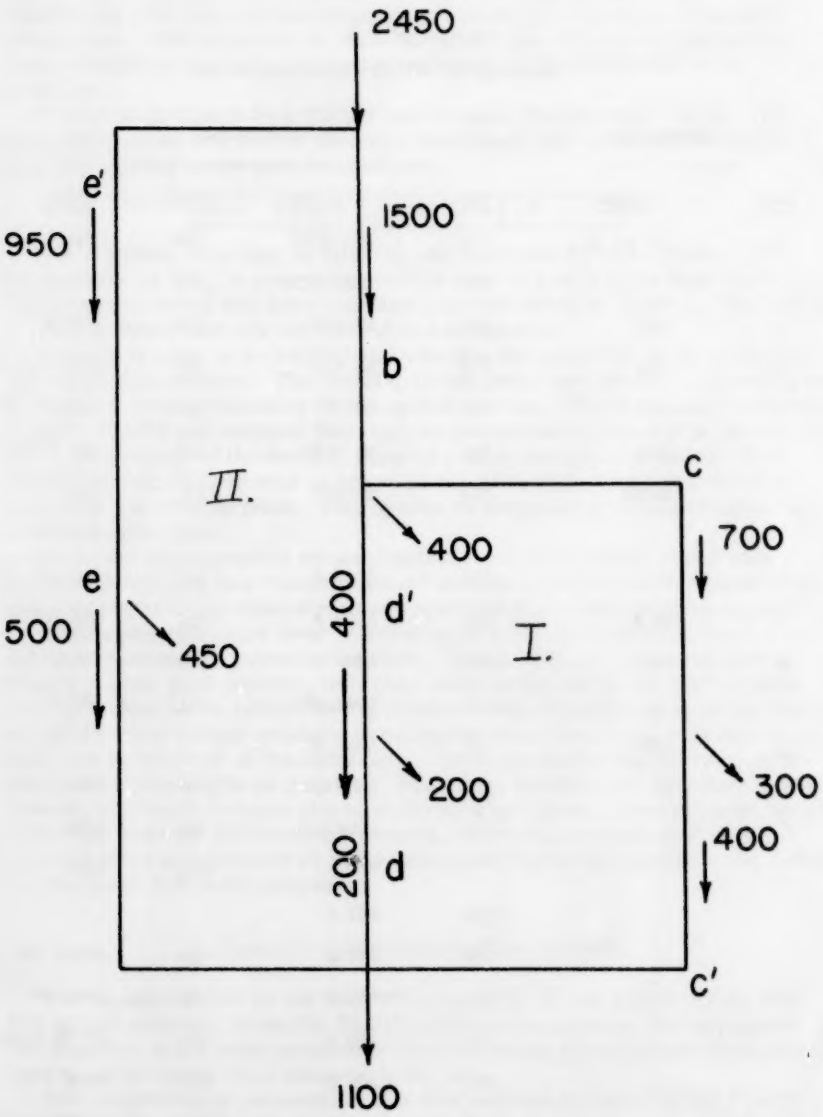
The programming and solution of a flow network by applying the FLOPS system to the modified Hardy Cross method of successive approximation is outlined as follows:

1. Preparation of flow diagram for machine operations
2. Initial program preparation
3. Program analysis and improvement
4. Program card punching and FLOPS deck acquisition

TABLE I

Summary of "FLOPS" Operation Codes

O_1 Operations		O_2 Operations			
Num.	Alpha	Num.	Alpha	Num.	Alpha
0	Go to O_2	000	UNC STOP	300	SQRT
		200	COND STOP	301	EXP E
1	ADD	201	TR SGN	302	LOG E
2	SUB	202	TR EXP	303	SIN R
3	MPY	203	TR	304	COS R
4	DIV	204	TR SUBR	305	ART R
5	NGMPY	205	TR OUT	350	ABS
				351	EXP 10
6	TR A	100	LOOP A	352	LOG 10
7	TR B	010	LOOP B	353	SIN D
8	TR C	001	LOOP C	354	COS D
		110	LOOP AB	355	ART D
9	MOVE	101	LOOP AC		
		011	LOOP BC	400	READ
		111	LOOP ABC	401	CONS
				410	PCH
		500	SET A		
		050	SET B	450	START TR
		005	SET C	451	STOP TR
		600	ADD A	452	ST TR ERAS
		060	ADD B	454	NOOP
		006	ADD C	800	COUNT
		700	SUB A	453	SWITCH
		070	SUB B		
		007	SUB C		



NETWORK WITH ASSUMED FLOWS

Figure 1.

5. Load cards in machine for solution
6. Read out answers and interpret results.

The FLOPS deck and wired panel are made up so as to perform certain special operations and contain certain data which may be useful in connection with the solution of a variety of engineering problems. It may be noted that the deck contains data such that the exponent and logarithm of a number may be obtained. Each function or procedure contained in the deck has an operations code for the 0_1 or the 0_2 operations. The functions or operations used in this particular analysis include: 1. the absolute value of a number; 2. the logarithm of a number; 3. multiplication; 4. the exponent for base ten; 5. addition; 6. looping; 7. division; 8. subtraction; 9. transfer; 10. conditional stop; and 11. punch out of the answer on cards.

In setting up and computing a flow network the conventions and definitions shown in Table 2 were used to facilitate the work.

TABLE 2

1. Proceed around each hydraulic circuit in the network in a clockwise direction.
2. When proceeding with the flow: K, Q, and H are positive.
3. When proceeding against the flow: K, Q, and H are negative.
4. Summation H/Q: Arithmetically.
5. Summation H: Algebraically.
6. Q correction or Delta: Opposite to summation H sign.
7. Q correction or Delta transferred to adjacent circuit: Reverses sign.
8. $H = KQ^{1.85}$ (from the Hazen-Williams formula).
9. H is the head loss in the pipe in feet.
10. Q is the rate of flow in gallons per minute.
11. K is a constant, depending upon the size and roughness of pipe.

The operations used to compute the rates of flow and the losses in head m may best be seen from Table 3 shown below. The values for K corresponding to various values of the Hazen-Williams C for circular pipes from 4 to 60 inches in diameter are given in Table 4.

A complete program for this particular problem together with the accompanying data will be found in Table 5. Fig. 2 shows the results of the computation. A more general program for extensive water distribution systems has been worked out, but it is not included here as it would not add materially to the present discussion.

The Analog Computer for Flow Networks

For purposes of comparison a brief statement with regard to the McIlroy analog computer for hydraulic networks will be included here. With the McIlroy computer as with the electronic or digital computer values of the

TABLE 3

Functions Desired and Required Machine Operations

Data Required: Values of K and assumed values of Q

Equivalent functions: $H/Q = KQ^{0.85}$ and $H = KQ^{1.85}$

Function	Machine Function and Name		Code Numbers	
1. $KQ^{0.85}$	Absolute value of Q	ABS	O_1	O_2
	LOG_{10} Q	LOG 10	0	352
	0.85 LOG_{10} Q	MPY	3	---
	$10^{0.85LOG_{10}Q}$	EXP 10	0	351
	$KQ^{0.85}$	MPY	3	Store answer
2. $KQ^{1.85}$	$KQ^{0.85}xQ$	MPY	3	Store answer
3. $\Sigma KQ^{0.85}$	(arithmetic)	ADD	1	Store answer
4. $\Sigma KQ^{1.85}$	(algebraic)	ADD	1	Store answer
5. $1.85 \times KQ^{0.85}$		MPY	3	Store answer
6. $\triangle$ $-\Sigma KQ^{1.85} / n\Sigma KQ^{0.85}$		DIV	4	Store answer
7. Correct Q or Q + $\triangle$ and repeat operation		ADD	1	Store in original Q location
8. Apply $\triangle$ to adjacent circuits or Q - $\triangle$		SUB	2	Store in original Q location

Hazen-Williams coefficient of resistance C must be known for each pipe in the system. Then the equivalent of the hydraulic system may be set up on the McIlroy computer in terms of electrical resistance. Superimposed voltage differences then may be placed on the system so as to obtain electrical currents into, out of and through the system corresponding to the hydraulic inputs and outputs. Measurements required would include voltage drops and current flow. These values must be reconverted into the corresponding hydraulic terms.

A Comparison of the Digital and Analog Computers for Flow Networks

The analog computer has the advantage, once it is set up for a particular system, of a simple procedure for altering current, voltage drop, and points of inflow and outflow. Also, a pipe or its equivalent may be altered or removed from the system very readily. No complicated arithmetic is involved as the hydraulic terms may be converted readily to their equivalent in electrical terms and the answers are converted with equal ease back to hydraulic terms. Savings in time are very readily obtained over the time required for solution by older methods.

One of the disadvantages of the analog computer is that a very expensive

piece of equipment must be purchased and maintained for the one special purpose of solving fluid network problems. Another disadvantage is that specialists for this equipment must be available to aid in setting up the equipment for the solution of each problem, and to aid in its operation. The engineer interested in the solution of the problem must spend considerable time in arranging his data for setting up the equipment on the analog computer and in converting hydraulic units into electrical units and then in converting the electrical answer terms into hydraulic units.

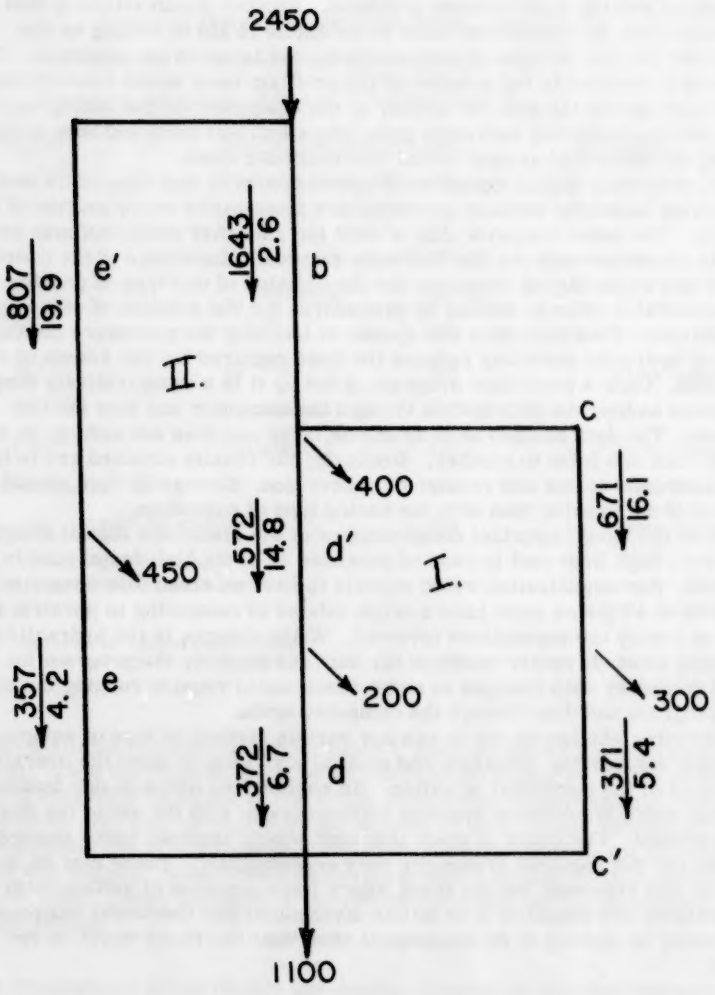
The electronic digital computer is advantageous in that computers suitable for solving hydraulic network problems are available in every section of the country. The same computer that is used for any other problems may be used equally advantageously for the hydraulic network. Knowledge of the procedure for the use of the digital computer for the solution of one type of problem is of considerable value in setting up procedures for the solution of other types of problems. Thus the effort one spends in learning the procedure for the solution of hydraulic networks reduces the time required for the set-up of other problems. Once a particular program is set up it is a comparatively simple procedure to feed the information through the computer and turn out the answers. The data utilized is in hydraulic units and does not need to be converted from one form to another. Similarly, the results obtained are in the same hydraulic terms and require no conversion. Savings in time should be as great if not greater than with the analog type of computer.

One of the most important disadvantages of the electronic digital computer is its very high first cost in case of purchase or of its high rental cost in case of rental. Any organization which expects to have an electronic computer available at all times must have a large volume of computing to perform in order to justify the expenditure involved. While changes in the hydraulic network may be made rather easily in the data and possibly the program for the digital computer such changes in some cases would require running the complete program and data through the computer again.

In deciding whether or not to use any certain method or type of equipment in solving engineering problems one must always keep in mind the overall economics of the particular situation. An engineering office in one locality analyzes water distribution systems arithmetically with the aid of the Hardy Cross method. The claim is made that high school students under supervision can turn out the required arithmetic very economically. There may be some merit to this argument but the trend where large amounts of arithmetical computations are required is to utilize mechanical and electrical equipment. This surely is proving to be economical otherwise the trend would be reversed.

Summary

The use of the McIlroy analog type computer was a distinct advance in the solution of water distribution system problems. The solution of water distribution system problems may be performed with the aid of the electronic digital computer with equal or greater facility. The digital computer has the additional advantage of being very useful in the solution of many other problems. This enhances any other advantages which it may have over the McIlroy analog computer. New developments may eventually alter the present devices and procedures for mathematical and engineering computations.



NETWORK WITH CALCULATED FLOWS & HEAD LOSSES

Figure 2.

TABLE 4

Values of "K" for 1000 ft. of pipe based on the Hazen-Williams formula

$h_f = K \times 10^{-5} \times Q^{1.85}$ where Q is rate of flow in gallons per minute

Pipe Dia. Inches / C	80	90	100	110	120	130	140
4	365	294	242	202	172	149	130
6	50.7	40.7	33.5	28.1	23.9	20.6	18.0
8	12.5	10.0	8.26	6.92	5.89	5.08	4.43
10	4.21	3.39	2.79	2.34	1.99	1.72	1.50
12	1.73	1.39	1.15	0.961	0.818	0.706	0.615
14	0.818	0.658	0.541	0.454	0.386	0.333	0.290
16	0.427	0.343	0.282	0.237	0.202	0.174	0.152
18	0.240	0.193	0.159	0.133	0.114	0.0980	0.0854
20	0.144	0.116	0.0953	0.0799	0.0680	0.0586	0.0511
24	0.0592	0.0476	0.0392	0.0329	0.0280	0.0241	0.0210
30	0.0200	0.0161	0.0132	0.0111	0.0094	0.00814	0.00710
36	0.00822	0.00661	0.00544	0.00456	0.00388	0.00335	0.00292
42	0.00388	0.00312	0.00257	0.00215	0.00183	0.00158	0.00138
48	0.00203	0.00163	0.00134	0.00112	0.000957	0.000825	0.000719
60	0.000683	0.000550	0.000452	0.000379	0.000323	0.000278	0.000243

TABLE 5

WATER DISTRIBUTION SYSTEM - 2 CIRCUITS

LOC	# WD	MNE	+ −	O_1	O_2/A	B	C	REMARKS
300	6	ABS	−	0	350	200	000	Absolute Value of Q
		LOG 10	+	0	352	000	000	LOG 10 Q
		MPY	+	3	000	998	000	$0.85x \text{ LOG}_{10} Q$
		EXP 10	+	0	351	000	000	$Q^{0.85}$
		MPY	−	3	000	100	930	$KQ^{0.85}$
		ADD	+	1	000	931	931	$\Sigma KQ^{0.85}$
306	6	MPY	−	3	930	200	000	$KQ^{1.85}$
		ADD	+	1	000	932	932	$\Sigma KQ^{1.85} = \Sigma h$
		LOOP B	+	0	010	004	300	
		MPY	+	3	931	997	000	$-1.85x\Sigma KQ^{0.85}$
		DIV	+	4	932	000	971	$\Sigma KQ^{1.85}/-1.85x\Sigma KQ^{0.85} = \triangle_1$
		ABS	−	0	350	204	000	Absolute Value of Q
312	6	LOG 10	+	0	352	000	000	$\text{LOG}_{10} Q$
		MPY	+	3	000	998	000	$0.85x \text{ LOG}_{10} Q$
		EXP 10	+	0	351	000	000	$Q^{0.85}$
		MPY	−	3	000	104	930	$KQ^{0.85}$
		ADD	+	1	000	933	933	$\Sigma KQ^{0.85}$
		MPY	−	3	930	204	000	$KQ^{1.85}$
318	6	ADD	+	1	000	934	934	$\Sigma KQ^{1.85} = \Sigma H$
		LOOP B	+	0	010	005	311	
		MPY	+	3	933	997	000	$-1.85x\Sigma KQ^{0.85}$
		DIV	+	4	934	000	972	$\Sigma KQ^{1.85}/-1.85x\Sigma KQ^{0.85} = \triangle_2$
		ADD	−	1	971	200	200	$Q + \triangle_1$ (1st loop)
		LOOP BC	+	0	011	004	322	
324	6	SUB	+	2	205	971	205	$Q_d' - \triangle_1$
		SUB	+	2	206	971	206	$Q_d - \triangle_1$

TABLE 5

(Continued)

LOC	# WD	MNE	+ −	O_1	O_2/A	B	C	Remarks
		ADD	−	1	972	204	204	$Q + \Delta_2$ (2nd Loop)
		LOOP BC	+	0	011	005	326	
		SUB	+	2	202	972	202	$Q_d - \Delta_2$
		SUB	+	2	203	972	203	$Q_d' - \Delta_2$
330	6	SUB	−	2	931	931	931	Clearing Sum
		LOOP ABC	+	0	111	004	330	Boxes
		MPY	+	3	999	971	971	Place Δ, in Last Mathematical Operation for Comparison with Tra. Exp.
		TR. EXP.	+	0	202	047	300	
		TRA A	+	6	995	336	336	
		COND. STOP	+	0	200	000	000	
336	5	ADD	+	1	996	995	995	
		MPY	+	3	999	972	972	
		TR EXP	+	0	202	047	300	
		PCH	+	0	410	200	208	
		COND. STOP	+	0	200	000	000	
		TRANSFER CARD						77,78,79
751	0		+					001

TABLE 5
(cont'd)

LOC	# WD	MNE	+ -	O_1	O_2/A	B	C	REMARKS
995	5		+	0	236	000	000	0236 For TRA. EXP.
			+	0	001	000	000	0001 Operation
			-	1	850	000	050	-1.85
			+	8	500	000	049	0.85
			+	1	000	000	050	1.0
100	6		+	9	490	000	045	$K_c = 9.49 \times 10^{-5}$
			+	9	490	000	045	$K_c' = 9.49 \times 10^{-5}$
			+	1	172	000	046	$K_d = 11.72 \times 10^{-5}$
			+	1	172	000	046	$K_d' = 11.72 \times 10^{-5}$
			+	2	900	000	044	$K_b = .2900 \times 10^{-5}$
			+	1	172	000	046	$K_d = 11.72 \times 10^{-5}$
103	3		+	1	172	000	046	$K_d = 11.72 \times 10^{-5}$
			+	8	280	000	045	$K_e = 8.28 \times 10^{-5}$
			+	8	280	000	045	$K_e' = 8.28 \times 10^{-5}$

								Ans.		H
200	6		+	7	000	000	052	Q_c	671	16.1
			+	4	000	000	052	Q_c'	371	5.4
			-	2	000	000	052	Q_d	372	6.7
			-	4	000	000	052	Q_d'	572	14.8
			+	1	500	000	053	Q_b	1643	2.6
			+	4	000	000	052	Q_d'	572	14.8
			+	2	000	000	052	Q_d	372	6.7
			-	5	000	000	052	Q_e	357	4.2
			-	9	500	000	052	Q_e'	807	19.9

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MAINTENANCE OF FINE BUBBLE DIFFUSION[a]

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(Proc. Paper 1609)

SYNOPSIS

As a result of a two year laboratory study at the State University of Iowa, it has been confirmed that diffuser media can be clogged externally by a number of specific materials in the tank liquor, and internally by particulate matter in the air supply. Further, it was found that all forms of external clogging are aggravated by particulate matter in the air supply.

Corrective measures are proposed for each of the causes of clogging but because particulate matter is the most important it is recommended that it be corrected first. The commonly accepted standard for air quality of 0.5 mg/1000 cu ft is shown to be unsatisfactory under certain conditions. A new standard of 0.1 mg/1000 cu ft is proposed. Plant operating records are presented to show that a clogging problem was eliminated after the installation of precoated bag filters producing air meeting this standard.

The corrective measures for other specific causes of clogging can be effectively applied only after the air supply is adequately filtered.

INTRODUCTION

The activated sludge method of sewage treatment with diffused air aeration has been in general use in the United States for over 35 years. Designing engineers recognized from the beginning that for the most economical oxygenation, air must be diffused into the mixed liquor in the form of fine bubbles. The fine bubbles present a maximum area of air-water interface and maintain a minimum velocity of bubble ascent through the water, thereby increasing the time available for oxygen transfer from the bubble to the tank liquor.

Fine bubble diffusion for activated sludge is commonly attained by discharging the air through porous ceramic tubes or plates, through saran cord

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wound tubes or through other materials with small pores which give fine bubble diffusion.

The clogging of diffuser media has presented an operating problem at some plants, but not at all. The problems of clogging have been so severe in some places that continuous plant operation was practically impossible. At others it has been largely a nuisance and the cost of cleaning diffusers at regular intervals has been evaluated against the cost of the additional air required if large bubble diffusers were accepted.

A comparison of operating experiences from various plants has often led to confusion as to the real cause of diffuser clogging. The cause of clogging must be established before clogging can be eliminated. This paper is presented to report on the results of a two year laboratory study sponsored by an aeration equipment manufacturer to establish various causes of diffuser clogging and to develop specific corrective measures.

As a result of these studies it has been shown that diffusers can be clogged with particulate matter in the air supply, with fine sand in the tank liquor, with excessive calcium bicarbonate hardness in the water supply, and with reduced iron salts in the incoming liquid. If one or more of these factors are recognized as being present either before a plant is built or in an operating plant remedial measures may be taken to avoid the specific difficulty so fine bubble diffusion can be used to obtain maximum oxygen absorption.

The Problem

For the successful operation of an activated sludge plant the head loss across the diffuser media must be maintained as a small per cent of the total head the blowers operate against so as not to reduce the quantity of air discharged or increase the input horsepower beyond the capacity of the prime mover. A serious clogging exists if the media clogs so rapidly that the cost of cleaning or replacing the media is greater than the cost of additional air required for large bubble aeration. The fact that a clogging problem does not exist at all installations indicates that the causative factors are not always present. This provides a clue to a possible solution of the problem.

The cost of the additional air required for large bubble aeration compared to the cost for fine bubble aeration has been a subject of discussion among designing engineers almost since the development of the activated sludge process. For many years this cost was difficult to evaluate because of lack of data from parallel operation and lack of research studies indicating the different absorption rates for air of various bubble sizes. Recent work by Ippen and Carver,[1] Carver,[2] King,[3] Haney,[4] Pasveer,[5] and others all clearly indicate the great advantage of small bubble aeration. The unanimity of the results, even though various techniques were used for the studies and for evaluating the results, leave little doubt as to the desirability of fine bubble aeration. These results are also in conformity with basic laws of physical chemistry.

The basic problem is then how to maintain the diffuser media so that fine bubble aeration can be more economically utilized.

Plant Experience

At many plants the cause of diffuser media clogging has been apparent and

corrective measures have been instituted. At others an examination of factors which might have caused the clogging led only to confusion.

Particulate matter in the air supply has most often been recognized as the cause of clogging. Initial clogging has been encountered where construction dirt was not removed from the mains and was blown into the media when the plant was placed in operation. Cumulative clogging has often been caused by particulate matter in the air which was present either because the filters were not adequate to remove the material or because of an operating failure.

Ferrous iron salts in the sewage, which oxidize to ferric at the diffuser media, created a clogging problem at such locations as Cleveland Southerly and Gary, Indiana, where industrial wastes were present. Other apparent causes, mentioned in Manual of Practice No. 5,[6] are oil from improper operation of viscous filters, rust or scale from air piping or oxidation of organic pipe coating, sewage solids entering the piping system through broken mains, organic solids or fine silt entering the media when the air pressure is off, precipitated deposits from industrial wastes and carbonates from hard water.

These latter apparent causes are only suggestions as there is no information indicating actual failure or quantitative data to give any indication of rates of clogging.

Laboratory Studies to Reproduce Plant Conditions

These studies were undertaken to reproduce clogging in the laboratory similar to that encountered in operating plants but under controlled conditions where quantitative data could be obtained. All laboratory studies can be classed as accelerated studies because the work was planned to clog the media in as short a time as possible and yet be comparable to plant conditions. In most cases the laboratory conditions are comparable to the worst possible field conditions.

Each laboratory clogging study was carried on with three one foot long diffuser tubes mounted along one side of an aeration tank having a width of 5-1/2 ft, a water depth of 4 ft and a length of 2-1/2 ft. The bottom and top edges of the tank were filleted to enhance the rolling action similar to that in conventional diffused air aeration tanks. Air was supplied by a 12 cfm rotary positive displacement blower, it was metered by an orifice to each individual diffuser tube. The quality of the air was controlled by the use of line-filters and the quality was determined by discharging a known volume of air through a Soxhlet thimble according to the procedure suggested by Beck.[6]

Once a test was started, operation was continuous to the end of the test period. Air rates and all pressure readings were recorded once a day and air flow to the individual tubes adjusted as necessary to maintain a constant flow of 3 cfm per one foot tube.

Iron Clogging

The primary object of this series of studies was to determine the effect of a ferrous salt in the aerating liquor on the rate of tube clogging.

For Run No. 2 and 3 there was a continuous flow of sewage through the tank to provide a detention period of about 1-1/2 hours. Ferrous sulphate was added to this flow to provide an average concentrate of approximately 100 ppm Fe (500 ppm $FeSO_4$ 7 H_2O) in the incoming sewage. To provide a control an

additional tank was utilized. It received a similar flow of sewage but did not receive iron salt. The diffuser units for these two runs were standard saran cord wrapped stainless steel core tubes.

In Run No. 2 the effect of high particulate matter in combination with iron was observed by filtering the air to only one diffuser. The average quality of the raw air was 4.7 mg of particulate matter per 1000 cu ft on the basis of a 60,000 cu ft sample. The results of the run are shown in Fig. 1. It is apparent that the iron was responsible for an exceedingly rapid rate of clogging and that at this rate of clogging the high particulate matter had no appreciable effect. It is also apparent that it was the iron salts and not the sewage since the control showed no increase in head loss.

Run No. 3 was conducted in exactly the same way as the earlier run except that head loss data were recorded at greater frequency and were compared with the actual hours during which the iron was added. The iron added over each 24 hour period provided an average of 100 ppm as iron in the incoming sewage but the period of actual addition was somewhat less than 24 hours. The hours of iron addition as well as head loss results are shown in Fig. 2. Cleaned, rather than new diffuser units, were used for this series but this apparently had little effect. The raw air had an average particulate content of 5.0 mg/1000 cu ft and the filtered air had 0.0 on the basic two 23,000 cu ft samples. The iron addition was not started until 2-1/2 days after head loss observations were begun. When the iron was started its effect was immediately apparent and each time it was temporarily stopped there was a partial recovery. It would also appear that the particulate matter in the raw air aggravates the clogging as indicated by the difference in head loss between No. 1 and No. 2 tubes even though it had no effect in this short period on the control.

Run No. 13, completed at a later date was to observe the rate of clogging of polyethylene winding in comparison with saran and to observe the effect of different quantities of iron additions. The procedure was the same as for the previous runs except that tap water was used instead of sewage and the flow provided for 4-1/4 hours detention instead of 1-1/2 hours. The ferrous sulfate was added to provide 100 ppm of Fe, based on the influent flow. The results of this run are shown in Fig. 3. There was no difference in the clogging rate between the saran and polyethylene. The data from this run was, however, of value when compared with Runs No. 2 and 3 (Fig. 1 and 2) where the iron concentration in the influent was the same but the quantity was different. If a tube can be considered clogged when the increase over the initial head loss is 1 lb per sq in, a comparison of time of clogging for the three runs can be made as indicated in Table I. The daily addition of iron for Runs 2 and 3 was 2.5 times the amount added for Run 13 and the time required for clogging in Run 13 was respectively 2.0 and 3.0 times the average length of time required for clogging the tubes in Runs 2 and 3. It would thus appear that the quantity

Table I

Run No.	Figure No.	Ferrous Sulfate Addition		Time required for tube to clog days		
		ppm Fe	$FeSO_4 \cdot 7H_2O$ lb per day	Tube No.1	Tube No.2	Average
2	1	100	25	4.9	4.1	4.5
3	2	100	25	2.5	3.5	3.0
13	3	100	10	9	9	9

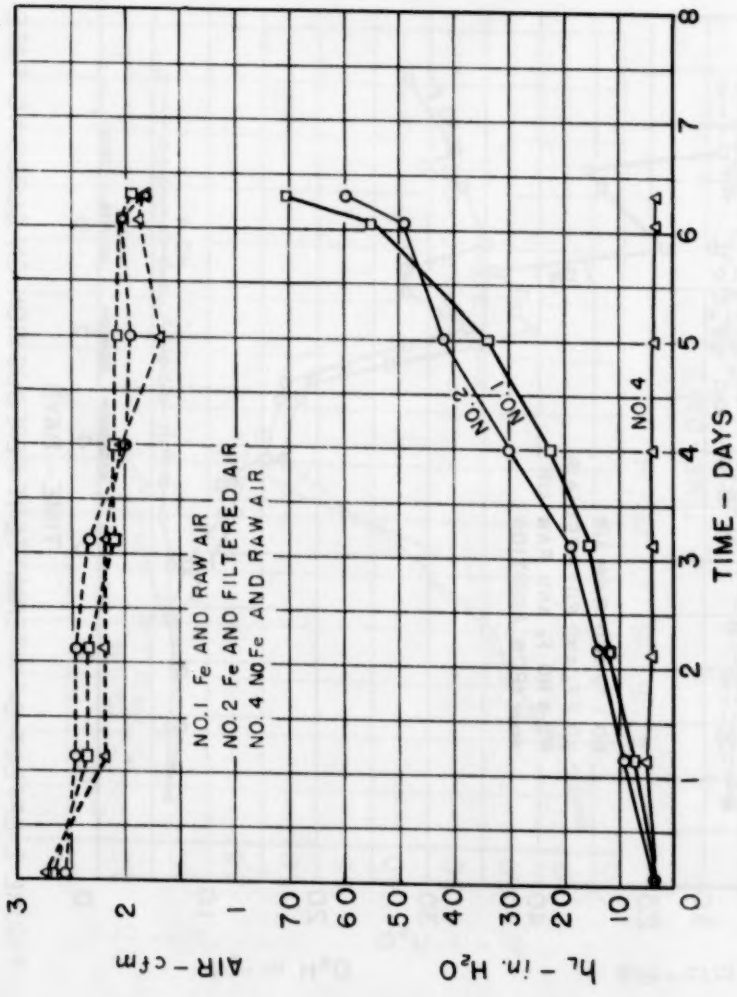

FIGURE 1 EFFECT OF FERROUS IRON SALTS ON CLOGGING RATE

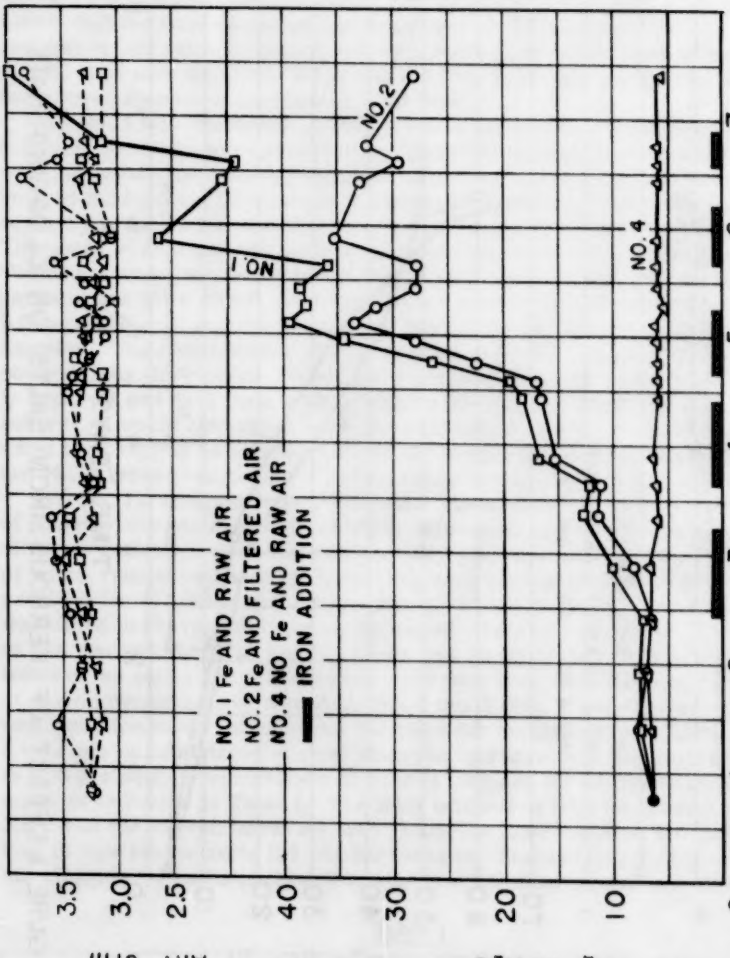

FIGURE 2 EFFECT OF INTERMITTENT FERROUS IRON ON CLOGGING RATE

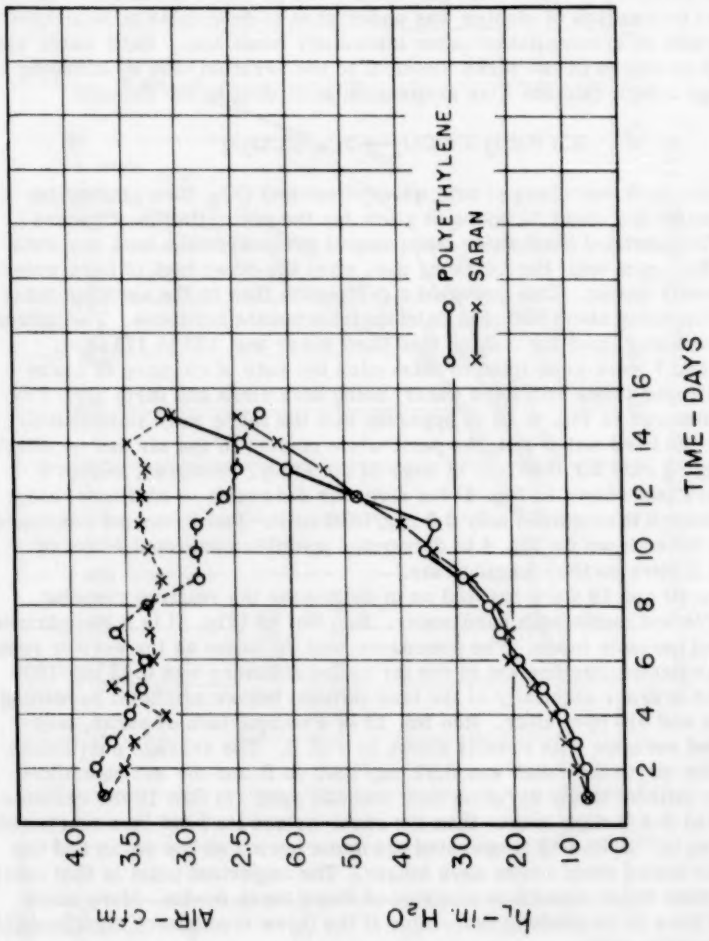

FIGURE 3. CLOGGING RATE OF SARAN AND POLYETHYLENE MEDIA IN PRESENCE OF FERROUS IRON.

of iron added is more important than the concentration. Particulate matter in the air supply may contribute to the clogging rate as indicated by Run 3 but these runs are too short to obtain conclusive results on this point.

Calcium Bicarbonate Clogging

Natural hardness in water has also been cited as cause for diffuser media clogging so this series of studies was undertaken to determine at what rate clogging could be accomplished under laboratory conditions. Hard water was artificially produced in two tanks identical to the aeration tank by diffusing CO_2 through a high calcium lime suspension according to the formula

$$3Ca(OH)_2 + 6\,CO_2 \longrightarrow 3Ca(HCO_3)_2$$

Each day one tank was charged with water, lime and CO_2, then allowed to stand quiescent for about 23 hours to allow for the precipitation of excess $CaCO_3$. The clarified hard water was pumped to the aeration tank at a rate of 0.15 to 0.25 gpm until the following day, when the other tank of hard water would be ready to use. This provided a continuous flow to the aeration tanks of water containing about 650 ppm calcium bicarbonate hardness. The natural alkalinity of water used for making this hard water was 100 to 175 ppm.

Runs 6 and 7 were completed to determine the rate of clogging of saran wrapped diffuser tubes with hard water, using both clean and dirty air. The results are shown in Fig. 4. It is apparent that the tubes were immediately affected by the hard water and, the particulate content of the air had no effect on the clogging rate for the first 11 days of the study. However, in Run 6 after 17 days (not shown in Fig. 4) the dirty air did result in additional clogging even though it contained only 0.8 mg/1000 cu ft. Run 7 was not continued beyond the time shown on Fig. 4 to determine possible additional effect of particulate matter on the clogging rate.

Runs No. 10 and 12 were carried on to determine the relative clogging rates of different media with hard water. Run No. 10 (Fig. 5) is a comparison of saran and ceramic tubes. The procedure was the same as the earlier runs. The average particulate content of the air to the diffusers was 0.43 mg/1000 cu ft and the average alkalinity of the tank influent before artificial hardening was 54 ppm and 610 ppm after. Run No. 12 is a comparison of saran, polyethylene, and ceramic with results shown in Fig. 6. The average particulate content of the air to the tubes was 0.32 mg/1000 cu ft and the average alkalinity of the influent to the aeration tank was 635 ppm. In Run 10 the ceramic tube operated 5-1/2 days longer than the saran before its head loss increased one lb per sq in. In Run 12 it operated the same period as the saran and the polyethylene lasted about seven days longer. The important point is that calcium bicarbonate water results in clogging of these three media. Many more tests would have to be made to determine if the three media have significantly different clogging rates.

The question of whether particles of precipitated calcium carbonate in the water causes clogging or if it is direct deposition of the calcium carbonate on the media was resolved in Run No. 10. For the first 13 days, after the start of the bicarbonate addition, all precipitated calcium carbonate was maintained in suspension in the aeration tank. After 13 days the aeration tank was operated so that all precipitated material was deposited in the tank bottom, as in

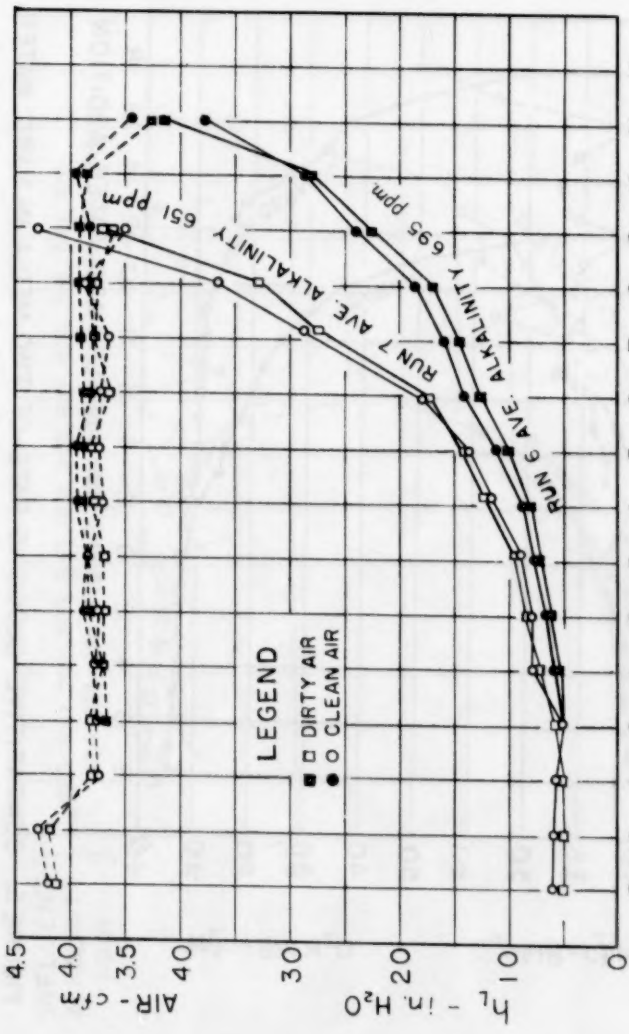

FIGURE 4. EFFECT OF CALCIUM BICARBONATE HARDNESS ON CLOGGING RATE.

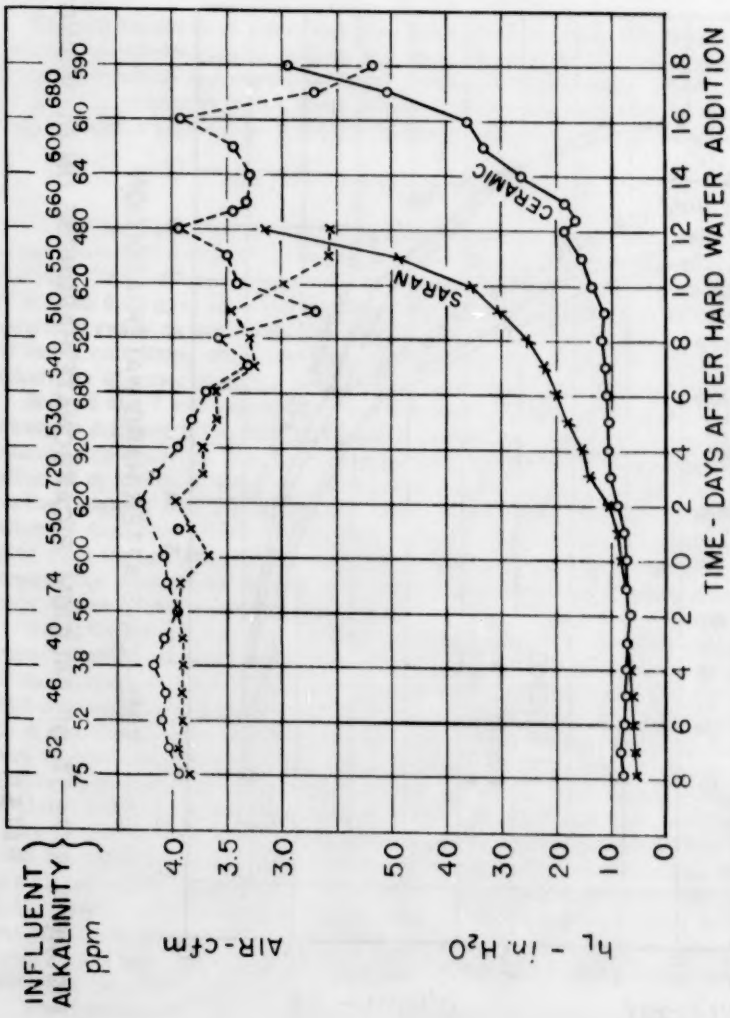

FIGURE 5. COMPARITIVE CLOGGING RATE OF TWO MEDIA IN HARD WATER.

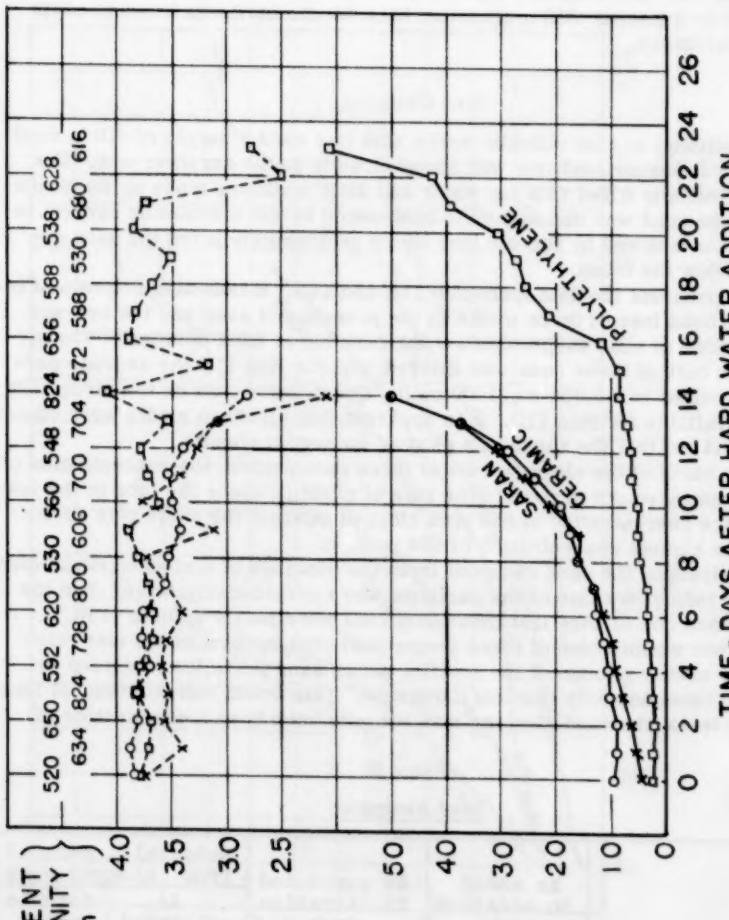

FIGURE 6. COMPARITIVE CLOGGING RATE OF THREE MEDIA IN HARD WATER.

a grit chamber, and the aerating liquor had zero ppm of suspended solids. An examination of Fig. 5 indicates the rate of clogging of the only tube operating (the ceramic tube), continued as before. It was concluded that the clogging must have been caused by direct deposition on the tube.

In considering these results in relation to natural hardness it must be remembered that this hardness was all calcium bicarbonate. A similar effect would not be expected with magnesium bicarbonate hardness because of its greater solubility.

Sand Clogging

In an attempt to clog diffuser media with fine sand a supply of silica sand having the following analyses was added directly to the aeration tank. The tank was initially filled with tap water and daily additions made up for evaporation. The sand was maintained in suspension by the circulating current in the aeration tank and by two air lifts which continuously lifted the sand deposited below the tubes.

Fig. 7 presents the data from Run 11C and 11D. It indicates the rate of increase of head loss of three media in the presence of sand and the average concentration of sand suspension for the duration of each tube test. The air supply for both of these runs was filtered, and for Run 11D the average particulate content was 0.034 mg/1000 cu ft. Quantitative data on the air quality are not available for Run 11C. It is apparent that all three media were clogged by the sand but that the saran was clogged somewhat sooner.

On the basis of the clogging rate of three saran tubes, the concentration of the sand appeared not to control the rate of clogging since the tube in the lowest average concentration of 658 ppm clogged at about the same rate as the tube in the highest concentration of 992 ppm.

An analysis of the sand removed from the windings of a clogged saran tube (Table II) indicates some of the particles were considerably larger than the original sand and further that these particles were partly soluble in HCL. Microscopic examination of these larger particles confirmed the fact that they were mostly clumps of the smaller silica sand particles cemented together, presumably with calcium carbonate. This would indicate that, at least for these tests, the sand clogging may actually have been a combination of

Table II

Sand Analysis

	As added to aeration tank	As suspended in aeration tank	Material removed from clogged tube	
			As collected	Soluble in HCl
over 250 microns	0.0%	0.0%	7.4%	15.9%
250-147 "	0.3%	0.1%	7.2%	22.7%
147-105 "	26.3%	34.8%	22.8%	10.2%
105-74 "	51.9%	47.5%	27.2%	5.9%
Less 74 "	21.5%	17.6%	35.4%	7.0%

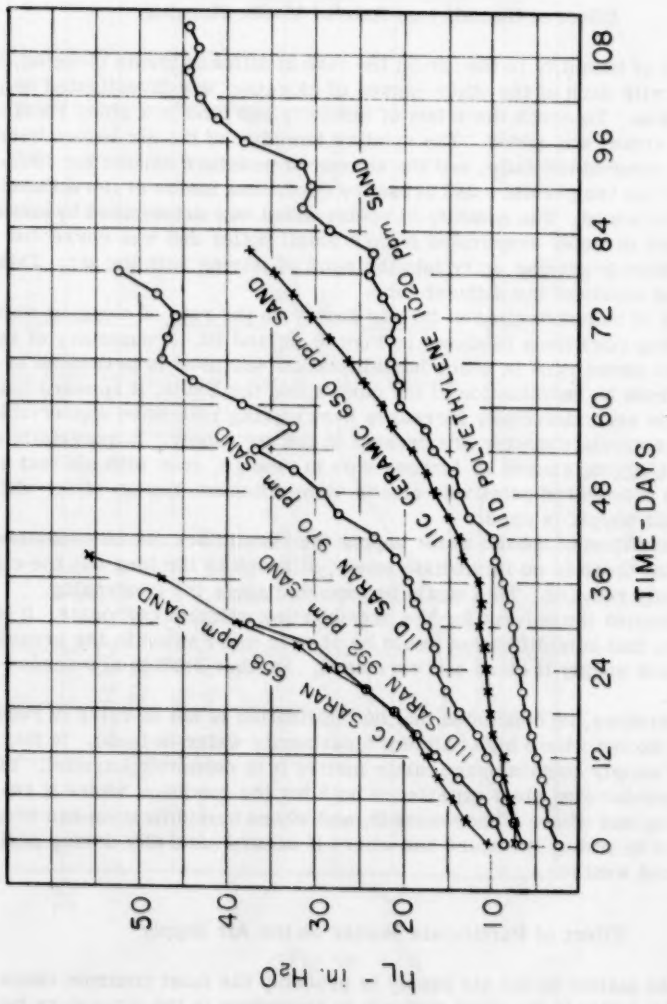

FIGURE 7. EFFECT OF FINE SAND ON CLOGGING RATE.

sand clogging and calcium carbonate deposition. Since the greatest proportion of the silica material removed from the windings, after correcting for the cemented particles, was under 74 microns, it can be assumed that it is the fine sand that causes the clogging. This may explain why sand clogging is seldom encountered in plants. This size is just not present.

Effect of Humidity on Rate of Media Clogging

The effect of humidity in the air on the rate of diffuser media clogging, in combination with each of the other causes of clogging, was investigated as part of other studies. To check the effect of humidity one tube in a study received air to which steam was added. The relative humidity of the air before humidification was determined daily, and the amount of moisture needed for 100% saturation at the temperature and pressure conditions inside of the diffuser tube was determined. The quantity of steam added was determined by measuring the amount of water evaporated from a small boiler and was corrected for condensation preceding entry into the point of mixing with the air. This point was just inside of the diffuser tube.

The effect of humidification of the air supply on the rate of clogging under three operating conditions is shown in Figs. 8, 9, and 10. A summary of results from all seven runs in which humidification was used is presented in Table III. From an examination of the curves and the Table, it appears that humidification very materially increases the clogging rate when appreciable amounts of particulate matter are present in the air supply. It materially increases the clogging caused by ferrous iron in sewage, even with air that until recently was considered relatively clean. Humidification has no effect when a truly clean air supply is used.

For the condition of a hard water supply the results are not as conclusive. It appears that there is no immediate effect, although in the long run the clogging is actually reduced. This would be expected since the condensing moisture becomes the solvent for the precipitating calcium carbonate. It is also expected that humidification would be of even more value in the presence of a hard water supply if clean air were used. Further studies are needed to confirm this.

It can, therefore, be concluded that humidification is not of value in reducing clogging except where high calcium bicarbonate water is found. In fact, when the air supply contains particulate matter it is definitely harmful. This generally corroborates plant experience both for the condition where it reduces clogging and where it increases it, and where humidification has been accomplished by adding moisture and where it occurs naturally during periods of warm humid weather.

Effect of Particulate Matter on the Air Supply

Particulate matter in the air supply is probably the most common cause of clogging and yet this is the most difficult to reproduce in the laboratory because of the difficulty of controlling the addition of these solids. Various dust feeding devices such as that recommended by the American Air Filter Institute for Test Air Filter[7] were considered but were rejected because they could not be adapted to the conditions required for testing diffusers nor could they be operated at a low enough rate of dust feed to be comparable to field

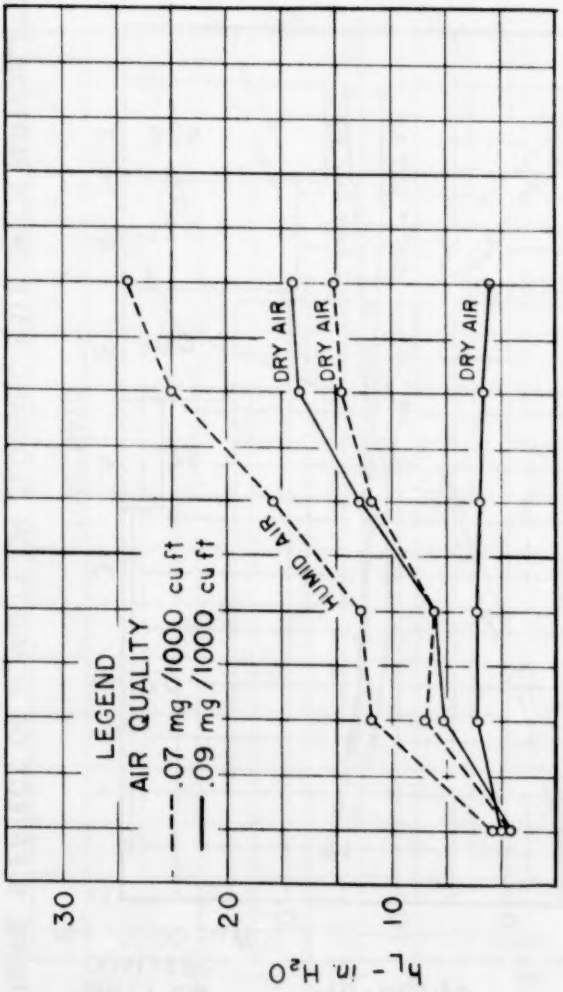

FIGURE 8. EFFECT OF HUMIDITY ON CLOGGING RATE WITH 100 *ppm* FERROUS IRON IN SEWAGE.

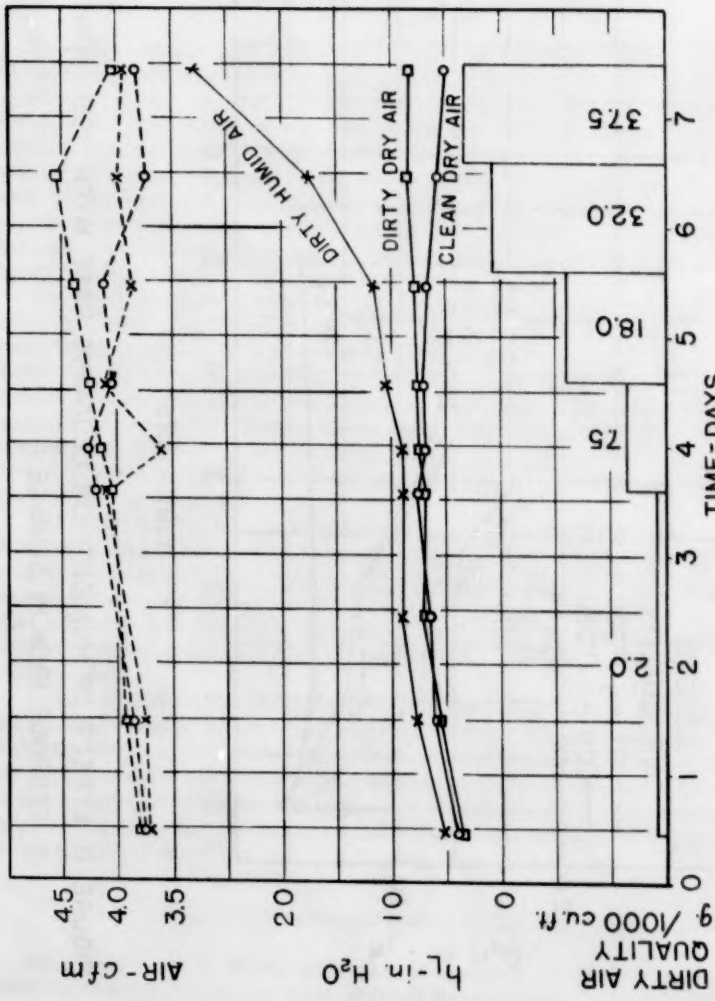

FIGURE 9. EFFECT OF HUMIDITY ON CLOGGING RATE WITH PARTICULATE MATTER IN AIR SUPPLY.

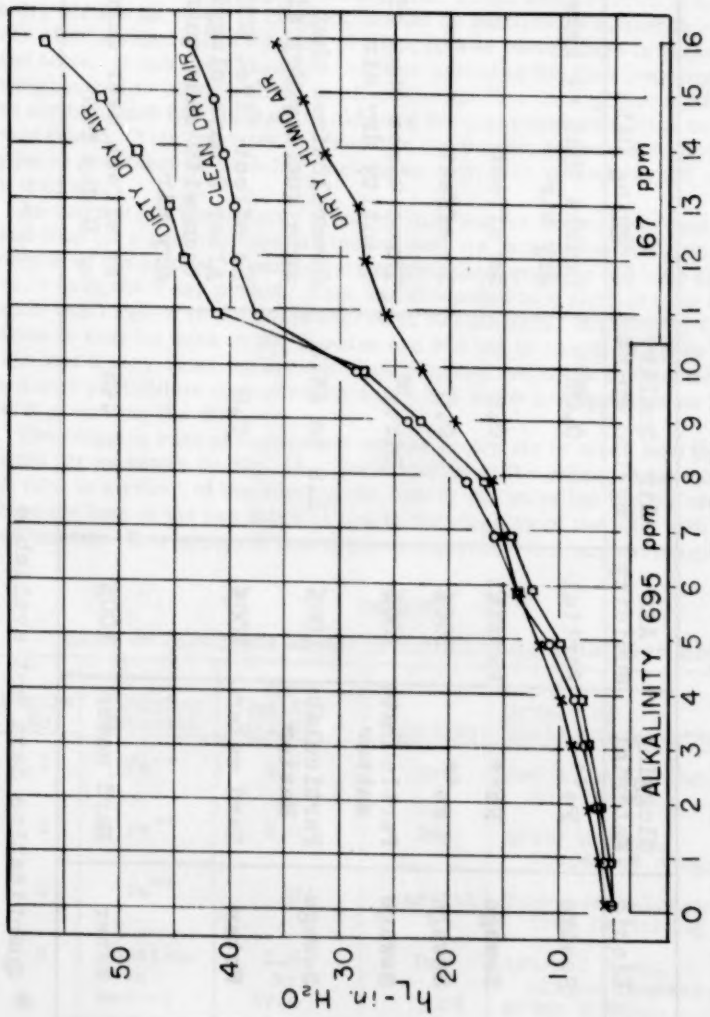

FIGURE 10. EFFECT OF HUMIDITY ON CLOGGING RATE WITH CALCIUM BICARBONATE IN WATER SUPPLY.

Table III

Effect of High Humidity Air on Rate of Clogging Diffuser Media

Run No.	Liquid	Clogging Material	Air Humidity	Air Quality mg/100 cu ft	Clogging Rate
1	Sewage	Fe^{++}	Partial	0.07	Double that at same quality dry air
2	Sewage	Fe^{++}	Partial	0.0	No effect
3	Sewage	Fe^{++}	100%	0.0	No effect
4	Sewage	Particulate Matter	100%	Dirty*	Three times that of same quantity dry air
5	Sewage	Particulate Matter	100%	1.8 to 43.3	Several times that of same quality dry air
6	Water	Hard water	100%	0.8	No effect first 9 days. After 9 days clogging less than with same quantity dry air
7	Water	Hard water	100%	.45	No effect over duration of 9 day run

* Quantitative data not available

conditions. The particulate feeder finally selected was a common kerosene lamp set at the blower intake. It could be adjusted to provide more or less smoke and thereby control the quality of the air. For preliminary studies the laboratory air was used as a source of dirty air with considerable success.

Particulate matter appears to be much more objectionable in air when it is not the only cause of clogging. Table IV is tabulation of the effect of particulate matter taken from Figs. 1 through 10. From these results it appears that in dry air the increase in clogging caused by particulate matter is effective only after an appreciable period of time, five to eleven days in these accelerated tests. It has been assumed that this period is the time required for enough pores to become filled to effect the air flow through the media. When the air is humidified the rate of clogging from particulate matter is greatly accelerated. This has been explained in the section on humidification. It also appears that there is no delay in clogging even with as little as .07 mg/1000 cu ft of air.

An estimate of the quantity of particulate matter required to increase the head loss (of a one foot tube) 22 inches over its initial loss was obtained by computing the amount of particulate matter discharged to the tube shown in Fig. 9 over the 7 day period. This was compared to a similar tube (Fig. 11) which was clogged to the same degree in 100 minutes. It required 875 mg of solids to clog the tube in 100 minutes and 555 mg to clog it in seven days. This was a very close agreement if corrections are made for the fact that with high particulate concentration there was some precipitation on the pipe walls preceding the diffuser.

The clogging rate of particulate matter in dry air is much less than in humid air as shown by Fig. 12. The variation of the pressure loss of the clean air tube is a result of the varying air flow to the tubes but the difference in pressure loss of the two tubes is due to the clogging of the tube with particulate matter. It is apparent that there is a partial recovery each time the

Table IV

Effect of Particulate Matter on Rate of Clogging Diffuser Media

Figure No.	Clogging Material	Air Quality mg/1000 ft	Humidity	Effect of Particulate Matter
1	Fe^{++}	4.7	Dry	For 6 days shown, no effect
2	Fe^{++}	5.0	Dry	After 5 days, increased clogging
8	Fe^{++}	.07	Partial	Increased clogging from beginning
9	Particulate Matter	1.8 to 37.4	Dry 100%	After 5 days, slight increase After 5 days, considerable increase
10	$Ca(HCO_3)_2$	.80 and .89	Dry	After 11 days, increased clogging

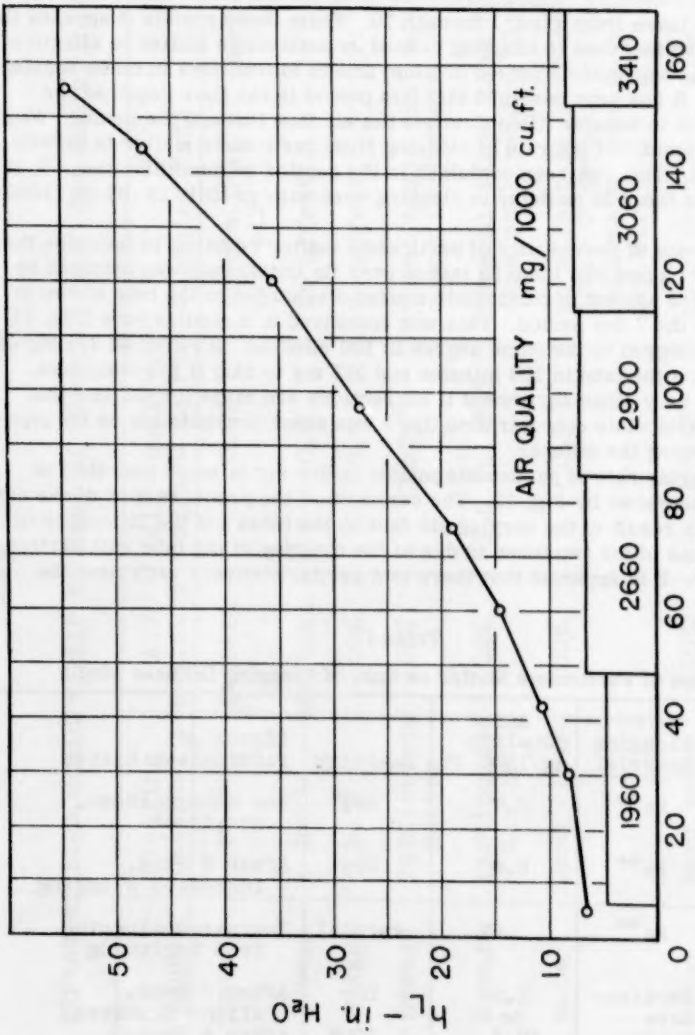

FIGURE 11. CLOGGING RATE WITH SATURATED AIR AND HIGH PARTICULATE CONTENT.

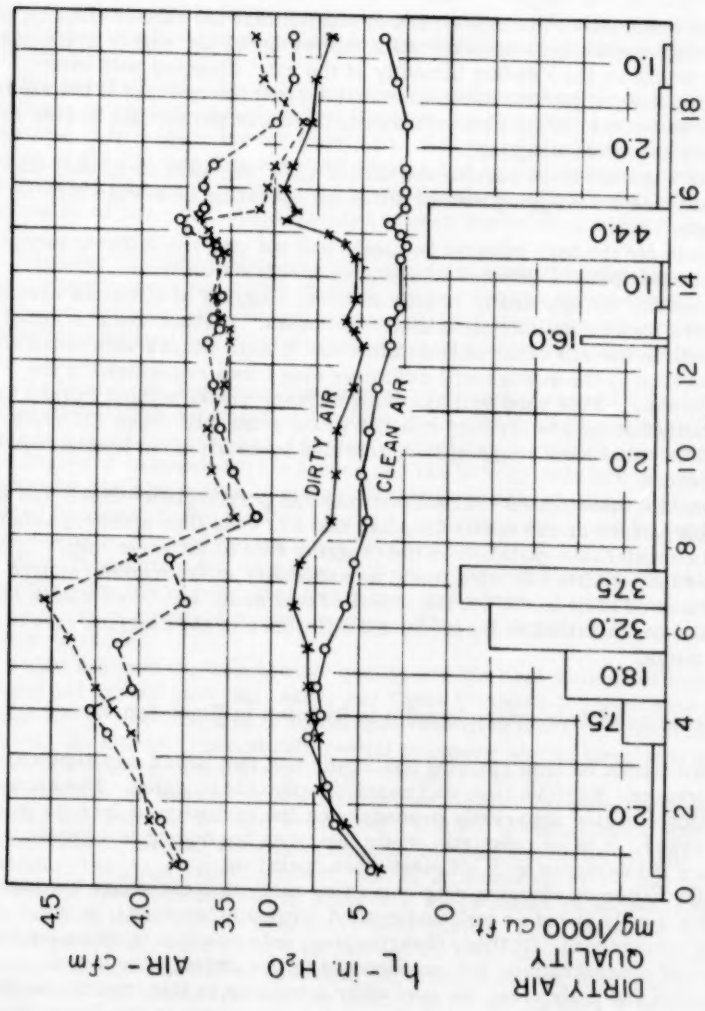

FIGURE 12. CLOGGING RATE WITH PARTICULATE MATTER IN DRY AIR.

particulate matter is reduced. Long time tests would be necessary to get complete clogging of a diffuser with dry air and with concentrations in the range of the test.

Summary of Laboratory Studies

It can be concluded from this series of studies that the rate of clogging of diffuser media resulting from particulate matter in the air supply is dependent to a large extent on the relative humidity of the air. Clogging with even moderate amounts of particulate matter may be extremely rapid if the air is saturated, whereas at 20% relative humidity the same particulate matter results in only moderate clogging.

Even very low levels of particulate matter (0.07 mg/1000 cu ft) will materially increase the clogging rate of diffusers operating in sewage high in ferrous iron.

With clean air the high relative humidity will not clog the diffuser media nor does it contribute to external clogging by ferrous iron.

High humidity air appears to reduce external clogging of diffusers operating in waters high in calcium bicarbonate hardness. Further work is necessary to confirm the full value of humidification in hard waters with clean air.

Ferrous iron in the sewage will definitely clog tubes regardless of the quality of the air. Fine sand will also clog diffuser media without regard to the air quality but apparently only specific grain sizes will cause difficulty. The possibility that sand clogging is aggravated by calcium carbonate cementing is indicated.

Of the media investigated—saran, ceramic and polyethylene—there was not a consistent pattern of one media clogging more or less than another, although there was considerable variation in the clogging rate of different media in the individual runs. Additional work would be necessary to determine relative clogging rates of each media but the object of this study was to eliminate the clogging and not to establish a percentage difference in the clogging rate of different media.

Discussion of Laboratory Results in Relation to Plant Experience

The information on iron clogging has confirmed that which was known from plant experience. Ferrous iron will result in external clogging. The elimination of this clogging apparently depends upon the exclusion of ferrous iron from the sewer. It is an industrial waste that does not belong in a public sewer since it interferes with efficient plant operation.

The information on clogging rate caused by calcium bicarbonate hardness in the water supply is not as well understood, probably because it is not commonly encountered. Of these bicarbonates, only calcium is objectionable and sufficient concentrations are encountered in few cities. The use of humidification and truly clean air may offer a solution to this type of clogging.

Sand clogging has been encountered in only a few plants in the United States apparently because the proper grain size sand was not present. The possibility of calcite cementing in connection with sand clogging should be investigated where sand is a problem. If it does contribute, corrective measures may be taken.

Clogging by particulate matter in the air supply is by far the most common

cause of clogging in operating plants and for this reason the information gained on this subject is most valuable. It has been shown that particulate clogging is indeed serious and furthermore that it is dependent to a large extent on the relative humidity of the air supply. This explains why clogging has been accelerated at many plants during periods of warm humid weather. It is not the humidity that clogs the media but rather the particulate matter, which is more objectionable in the presence of moisture. This work indicates the importance of providing clean air for air diffusion since dehumidification is not practical.

The value of clean air has previously been pointed out by others but there has been no general acceptance of the need for clean air nor any general acceptance of how clean the air should be. Anderson[8] pointed out in 1950 that plates at the Chicago Southwest Plant supplied with air cleaned to 0.5 mg/1000 cu ft lasted five times longer than plates supplied with 2.5 mg/1000 cu ft air. Operation of this plant since then has proven the value of using even cleaner air.

When the Chicago Southwest Plant was first started particulate material in the air supply resulted in the diffuser media clogging so severe that at times plant operation was almost impossible. In 1950 this plant pioneered the installation of precoated bag filters to provide truly clean air and solved their clogging problem.

The first units are described in their 1950 annual report[9] as follows: "This installation consisted of 12 units, each with four filters with 80 bags. Each bag is 17 feet long and 8 inches in diameter, closed at the upper end. The bags are suspended from a rack and the bottom ends are expanded in the separator plate by means of a steel spring sewn into the fabric. Dust is removed by vibrating the bags by means of a shaking device. The bags are lined with asbestos "floats" before going into service."

Beck[9] has provided operating data from the Southwest Plant which shows the efficiency of the precoated bag filters and proves the value of using truly clean air to eliminate diffuser clogging. This information is summarized in Tables V and VI.

Table V shows the average air quality for the first year after installing the filters and for the past two years, and Table VI shows the head loss data for plates in one aeration tank in continuous operation since the installation of the filters. An almost constant differential pressure across the plates for over seven years is certainly proof of the value of clean air.

A smaller but similar type air filter installation has been completed at the Mishawaka, Indiana, Sewage Treatment Plant for aeration tanks equipped with saran diffuser media. Although complete information on air quality is not available, the plant superintendent, Mr. R. A. Kronewitter[10] has provided the information summarized in Table VII, which shows no increase in head loss through the diffusers over a two year period. (The increase in the blower discharge accounts for the 0.1 lb. per sq. in. increase in blower pressure.) Because Mishawaka had no operating experience without high quality air filtration and has not determined the quality of its air its data may not prove anything but it does corroborate the experience at Chicago Southwest and the results of this experimental study as to the value of high quality air filtration.

Table V

Air Quality

Chicago Southwest Sewage Treatment Plant

From Annual Reports[9]

	Air Dust Concentration mg/1000 cu ft		
Year	Raw Air	Filter Discharge	As Supplied to Aeration Tanks
1951	3.71	.09	0.17
1955	10.9	.04	.04
1956	9.0	.014	.032

Table VI

Chicago Southwest Sewage Treatment Plant

Head Loss Across Diffuser Plates[9]

Tank No. 2, Battery B

Date	Cumulative Air Flow Per Plate Million cu ft	Rate of Air Flow Per Plate cfm	Plate Pressure Loss	
			At Tank Influent lb/sq in	At Tank Effluent lb/sq in
January 1950	5.0	3.19	.23	.21
March 1957	23.0	3.20	.21	.28

Table VII

Mishawaka, Indiana, Sewage Treatment Plant Record of Blower Discharge Pressures

Date	Outside Temperature °F	Blower Discharge Temperature °F	Blower Discharge cfm	Blower Discharge Pressure lb/sq in
January 1, 1955	36	126	2000	6.2
January 1, 1956	18	116	2100	6.2
January 1, 1957	20	115	2100	6.3
February 1, 1957	25	121	2200	6.3

Air Sampling

Because of the importance of clean air in preventing diffuser media clogging it is important that air samples truly represent the air quality. The air sampling technique proposed in the Manual of Practice No. 5[6] served as a basis for designing the air samplers for this experimental work. Other publications have shown the importance of further precuations which must be taken for sampling under the conditions existing in a sewage plant where the sample is a small per cent of the total air flow. Isokinetic sampling, where the sample is withdrawn through the sampling tube at the same velocity and in the same direction as that in the main stream, is necessary for accurate sampling according to Silverman.[11] It is pointed out that "various investigators differ as to the magnitude of the error produced by sampling at other than isokinetic velocity but agree that it is significant for particles greater than 5 microns." Stairmand[12] further discusses the importance of establishing uniform distribution of the particles across the conduit at the point of sampling. He installed a disk baffle in the center of the pipe approximately three pipe diameters upstream from the sampling point. Although Stairmand recommended the area of the disk should be 50% of the area of the pipe, he has indicated the possibility of using a smaller disk baffle to reduce the head loss. Fig. 13 represents a proposed sampling installation for determining the air quality for aeration tanks. A chart can be prepared for any given installation to show the flow rate required through the sampler to maintain isokinetic sampling for various total flow rates through the air main. If the proposed sampler is located where its temperature drops below the dew point of the sampled air, so the Soxhlet thimble is moist, the sampler and connecting piping can be enclosed in an insulated cabinet.

The cabinet can be heated above the dew point with a small thermostatically controlled electric heater or by attaching it with an open side to the warm air main. The Stairmand disk results in a uniform distribution of the air across the conduit so the center sampling point will truly represent the total air flow. For accurate results the sampling period must be long enough to obtain a

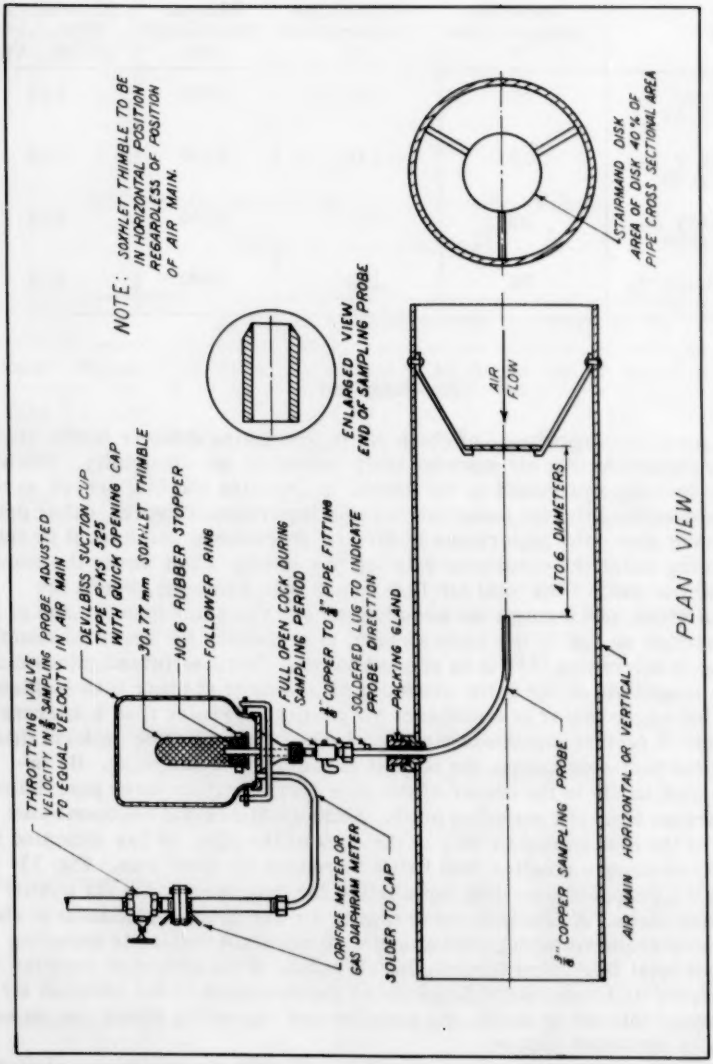
THROTTLING VALVE
VELOCITY IN ¾" SAMPLING PROBE ADJUSTED TO EQUAL VELOCITY IN AIR MAIN
DEVILBISS SUCTION CUP TYPE P-KS 525 WITH QUICK OPENING CAP
30 x 77 mm SOXHLET THIMBLE
NO. 6 RUBBER STOPPER
FOLLOWER RING
FULL OPEN COCK DURING SAMPLING PERIOD
¾" COPPER TO ¾" PIPE FITTING
SOLDERED LUG TO INDICATE PROBE DIRECTION
PACKING GLAND
ORIFICE METER OR GAS DIAPHRAM METER
SOLDER TO CAP
NOTE: SOXHLET THIMBLE TO BE IN HORIZONTAL POSITION REGARDLESS OF POSITION OF AIR MAIN.
ENLARGED VIEW END OF SAMPLING PROBE
AIR FLOW
4 TO 6 DIAMETERS
¾" COPPER SAMPLING PROBE
AIR MAIN HORIZONTAL OR VERTICAL
STAURMAND DISK AREA OF DISK 40 % OF PIPE CROSS SECTIONAL AREA
PLAN VIEW

FIGURE 13. SAMPLING APPARATUS FOR ISOKINETIC AIR SAMPLING FROM AIR MAINS.

significant weight of particulate material and the precautions recommended in Manual of Practice No. 5[6] should be followed to avoid weighing errors.

SUMMARY AND CONCLUSIONS

Air diffusion media has been clogged in the laboratory with particulate matter in the air supply, with fine sand in the tank liquor, with excessive calcium bicarbonate hardness in the water supply and with ferrous iron salts in the sewage.

As a result of these studies specific corrective measures are proposed which will allow the use of fine bubble diffusers, without clogging, for maximum oxygen absorption efficiency.

Particulate matter in the air supply has undoubtedly caused more media clogging in operating plants than any other single cause. It has been shown that the clogging rate from particulate matter is proportional to the quantity of the material in the air supply, and is dependent to a large extent on the relative humidity of the air supply. Plant operating records have shown that bag filters precoated with asbestos "floats" will supply air containing less than 0.10 mg of particulate matter per 1000 cu ft of air. With this high quality air, clogging from particulate matter will not be a problem. It is recommended that precoated bag filters be used for filtering air for aeration tanks and that they be specified to produce an air containing not more than 0.1 mg of particulate matter per 1000 cu ft.

Details of a sampling apparatus are shown which will provide true isokenetic sampling from air mains so samples obtained will represent the air flowing to the diffusers.

Humidification of the air supply is objectionable with dirty air but appears to reduce the external clogging of diffusers operating in waters high in calcium bicarbonate hardness. Further work is in progress to confirm the effectiveness of this procedure with clean air.

Ferrous iron, when present in the tank influent in sufficient concentrations, will definitely clog diffuser media. It should be excluded from the sewers as an undesirable industrial waste.

Fine sand of 100 microns diameter or smaller will clog diffuser media. The clogging rate may be aggravated if hard water is present. If so, corrective measures may be undertaken. If sand alone is responsible, the media pore size can be changed to avoid the particle size encountered.

All forms of clogging investigated in this study were aggravated by particulate matter in the air supply. It is, therefore, important that air quality be carefully examined, and if necessary that high quality air filters be provided before other corrective measures are undertaken. The corrective measures outlined above for specific causes of clogging can be effectively applied only with a clean air supply.

ACKNOWLEDGEMENT

These studies were carried on in the State University of Iowa Sewage and Industrial Waste Laboratory and were financed by a research grant to the University from the Chicago Pump Company of Chicago, Illinois.

Mr. Miles Lamb, Manager of Sewage Equipment Engineering Department,

Chicago Pump Company, provided information from their files on clogging problems in operating plants and served as a consultant in planning the individual studies so the results could be correlated with plant conditions.

Peter J. Neuspiel, Edgar A. Jeffrey and David A. Long, research associates at the State University of Iowa during the two year period of this study, conducted most of the laboratory work and wrote the thirteen progress reports which provided the basic data for this report.

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Paper 1610 April, 1958 SA 2

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SED RESEARCH REPORT NO. 16

On	REFUSE COMPOSTING EXPERIENCE IN THE NETHERLANDS
By	The Sanitary Engineering Research Committee, Solid Wastes Section
From the Data of:	W. A. G. Weststrate, Director
Acknowledgement:	The data presented herein represents the experience of the "V.A.M." Company as reported by Mr. W. A. G. Weststrate, Director.

SYNOPSIS

The Netherlands has successfully developed large scale municipal refuse composting through the "V.A.M." Company, a government financed agency. Essential research to reduce the cost of compost processing and to improve the quality of the compost product has been successfully carried on for many years. The results of the Netherlands pioneering work are critically reviewed.

INTRODUCTION

City refuse has been composted in The Netherlands and used for agricultural humus since the middle ages; this method of municipal refuse disposal was the most common method historically employed up until about 1900 when the introduction of chemical fertilizers reduced the demand for organic manure. The chemical fertilizers were less expensive and easier to handle because of their small volumes than the comparable large volumes of compost humus. This general situation still prevails in most agricultural regions. Even today, The Netherlands employs more artificial fertilizer per acre of arable land than any other country in the world.

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Currently in The Netherlands, about 25 per cent of the municipal refuse is incinerated, 50 per cent is disposed of through land fill, while the remaining 25 per cent is still processed into compost manure.

In 1931, The Netherlands Government, in order to promote the use of refuse composting, organized the Government Company entitled The "V.A.M." Refuse Disposal Company, Limited. The "V.A.M." contracts with certain municipalities to take their refuse; it then processes it into manure and sells it to the farmers. The various municipalities who thus dispose of their refuse pay the "V.A.M." Company about 8 to 36 cents per year per inhabitant depending upon transportation costs. This payment is reported to approximate the cost of alternative refuse disposal by incineration or land fill. The municipal contracts are entered into for a period of 30 years and the "V.A.M." Company must accept all municipal refuse during that time. The salvaged by-products such as iron and non-ferrous metals also help pay part of the compost operation costs. The "V.A.M." Company is self-supporting as a result of the total income obtained from municipal payment, by-product recovery revenue and the sales of the final compost product.

Discussion

Municipal refuse disposal has been completely mechanized by "V.A.M." at its two largest plants (Wijster and Micrlo). Refuse is conveyed each day by special train, completely enclosed and emptied automatically at the composting site. Refuse is then moistened and left to ferment at one plant for a period of four to eight months. After the refuse is decomposed, the rough compost is conveyed to a separating device termed a "Rasping Machine" wherein the finer particles are screened out and the larger organic clumps are broken down and reduced in size to produce a uniform product. The paper, glass, metal and other salvagable materials are further segregated and the remaining organic constituents which cannot be used for compost are disposed of to waste land.

At other treatment works normally operated by the local municipality an alternative composting method employs windrowing of refuse; this system requires a minimum of six weeks for completing the fermentation and composting of the refuse to a C/N ratio less than 20. There are no sanitary problems, such as odors, flies and rats present with the windrowing method.

The screening and "Rasping Machine" was developed by The "V.A.M." and consists chiefly of a large round plate, provided with holes, and a rough surfaced face. A number of arms revolve above the plate and rub the refuse over it so that fine particles are graded off and fall through the holes. Pieces of glass and clinker are removed and a uniform compost product is produced which is acceptable for use by gardeners, farmers and others.

The farmers apply about 13 to 17 tons of compost for each acre of land every four years. The chemical analyses of a typical compost product is as follows:

Organic matter (excluding non-combusted coal particles)	4.5 - 12%
Nitrogen	0.4 - 0.5%
Phosphate, soluble in mineral acid	0.4 - 0.5%
Potassium, soluble in water	0.2%
Calcium oxide, soluble in mineral acid	3.0%
Magnesium oxide, ditto	0.3%

Carbon dioxide	1.5%
Moisture	30%
Loss on ignition	18 - 25%

The Netherlands agricultural land contains considerable sandy type soils, deficient in humus content, and so the addition of compost is particularly valuable for them. The Agricultural Advisory Service for The Netherlands Government has assisted in the agricultural education of The Netherlands' farmers and market gardeners and has been instrumental in promoting the wide acceptance of the compost manure products.

A further use for the composted refuse in The Netherlands has been as a heating manure. An extra winter crop can be obtained by resorting to the growth of plants protected from freezing during cold winter weather by the heat generated during compost fermentation. The heating manure is placed in boxes covered with glass lying horizontally one to two ft. above the earth.

Further investigations are desirable to: (1) Improve the compost treatment efficiency, (2) Ascertain the influence of various carbon-nitrogen ratios on plant growth and nitrogen fixation, (3) Study the value of using compost in conjunction with artificial fertilizer additives, (4) Learn the value of composting refuse and sewage sludge together, and (5) Determine the true economic value of such municipal compost.

CONCLUSIONS

The Netherlands compost operations have resulted in a realistic method for collection and processing municipal refuse for agricultural purposes. The development of a practical system for materials handling, processing, packaging and distribution represents an outstanding achievement in solid waste engineering.

Credit

This research report is one of a series of professional contributions by the Committee on Sanitary Engineering Research,

E. R. Hendrickson	
W. T. Ingram	Air Pollution
Milo A. Churchill	Stream Pollution
R. W. Bogan	Sewage
C. H. Hull	Water
H. A. Faber	Public Health
R. Stone	Solid Wastes
Chairman, N. L. Nemerow	Industrial Wastes

has been prepared by the Solid Wastes Engineering Section.

John Merrell, Jr.
Leo Weaver
Frank Bowerman
Ralph Stone, Head

Paper 1611 April, 1958 SA 2

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SED RESEARCH REPORT NO. 17

On	COLOR REMOVAL FROM AZO DYE WASTES
By	The Sanitary Engineering Research Committee, Industrial Wastes Section
From Research Data of	Nemerow, N. L. and Wilson, W. L.
Acknowledgment	The Sanitary Engineering Division gratefully recognizes the generosity and professional courtesy of the original researchers in making their data available to the Society for review, presentation, and comment by the Industrial Wastes Section of the Research Committee.

SYNOPSIS

Azo type dye colors are used to a large extent in the textile and related industries. The efficiencies and economics of treating ten typical azo dyes with stannous chloride for color removal have been determined. Color may be removed by heating the dye waste (212° F.) for about two hours in the presence of a small quantity of stannous chloride, a powerful reducing agent.

Color Removal from Azo Dye Wastes

Introduction

When color is present in any industrial effluent, it signifies to the public that pollution exists. It, therefore, is apparent to industry and stream regulatory agencies that color should be removed from plant effluents. Intangible

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benefits arising from an investment expended on color removal include—(1) prevention of law suits, (2) improvement of public relations, and (3) increase in value of the properties adjoining the river into which the waste flows: In previous studies[1,2] the theories of color formation and means of removal were discussed.

Over 50 per cent of all the dyes consumed by the textile and other industries is of the azo type. One very promising method for removing color from azo type dyes involves the reduction of the nitrogen to nitrogen (-N = N-) bond with stannous chloride. The degree of color removal and the economics of the reduction are of utmost importance to the industry discharging colored wastes.

Procedure

Two-liter samples of each azo dye were prepared with dye concentrations similar to those used in actual industrial production. The wave lengths of maximum and minimum light transmittance were determined using a Coleman Universal Spectrophotometer, Model 14. The optimum dosage of stannous chloride required to achieve complete color reduction was determined for each dye by setting up a series of similar dye solutions at various $SnCl_2$ dosages. Two samples of 2000 ml each of dye were heated to 212° F. The optimum dosage of stannous chloride was added to one sample. The temperature and volume were maintained constant throughout the heating of both samples, and the time required for complete color reduction was measured. Final transmittances of the heated samples were measured only at the maximum and minimum wave lengths. The complete visible spectrum (400-700 mu) was run on the heated and reduced sample.

Results

Stannous chloride proved to be an effective reducing agent for removing color from azo dyes. It appeared to be equally effective with monoazo, disazo, and trisazo type dyes. Color removals ranged from 96 to 99.5 per cent at the wave length of maximum absorption on dyes treated with heat and stannous chloride. No shifting of dominant wave length was caused by this treatment. Heating alone reduced the color concentration from one to nine per cent in five of the ten dyes. With the other five dyes heat had no effect on color concentration.

A greater quantity of stannous chloride was required generally for disazo dyes than for monoazo types. Similarly, more reducing agent was used for trisazo dyes than for the disazo types.

The average time required to achieve complete color reduction generally increased as the number of (-N = N-) azo groups increased. It averaged about two hours.

The cost of $SnCl_2$ for each pound of dye in the waste averaged $2.38 for monoazo dyes, $6.95 for disazo dyes, and $12.91 for trisazo dyes. When these costs are projected to a basis of 100 pounds of goods dyed they are $0.165, $0.238, and $1.78, respectively.

Salt (NaCl) in the dye waste solutions caused a slight decrease in the percentage of color removal when treated with $SnCl_2$.

Engineering Implications

Since dye waste treatment results in an expenditure of money with seldom

TABLE 1

Color Removal Efficiencies with $SnCl_2$

Type Dye	$SnCl_2$ Reqd (mg/ mg dye)	Time Reqd. (Hours)	Color Rem. Per cent	Chem. Cost ($SnCl_2$) lb. Dye in Waste Dollars	[1]Chem. Cost/ 100 lb. Goods Dyed Ave. Shade Dollars
Monoazo					
1. Methyl Orange	3.0	1.0	98.5	2.38	0.0928
2. Erie Pink 2B	3.0	2.0	99.5	2.38	0.238
Avg.	3.0	1.5	99.0	2.38	0.165
Disazo					
3. Calcomine Diazo Black BHD	2.0	1.0	97.0	1.59	0.095
4. Direct Blue RW	4.0	1.5	99.5	3.18	0.095
5. Diphenyl Fast Red B 125%	25.0	3.5	98.0	19.87	0.745
6. Niagara Sky Blue 6B 200%	4.0	1.0	99.0	3.18	0.018
Avg.	8.75	1.75	98.4	6.95	0.238
Trisazo					
7. Erie Brown CN 200%	25.0	2.0	96.0	19.87	2.190
8. Erie Fast Brown GR 200%	10.0	2.75	99.0	7.95	0.596
9. Erie Green MT 150%	5.0	2.0	99.5	3.97	0.149
10. Calcomine Black 200%	25.0	1.5	99.0	19.87	4.170
Avg.	16.2	2.06	98.4	12.91	1.780

[1] Dye bath concentrations and exhaustion figures taken from Direct Dyes on Cotton published by E. I. du Pont de Nemours.

any direct return, the economics of the method employed assumes a leading role.

A small mill dyeing 1000 pounds of cottom per day using a 50-50 mixture of mono and disazo dyes would pay $2.01 per day for chemicals to remove the color from the wastes. Since the dye baths are normally very hot, the steam costs to maintain the elevated temperature for two hours would be a minor item. The economics of color removal with this process are more favorable if dyes are selected that can be used in very small quantities and are highly exhaustible. Fortunately these dye qualities are also required from a production standpoint.

No sludge results from this treatment.

Credit

This research report, which is one of a series of professional contributions by the Committee on Sanitary Engineering Research,

W. T. Ingram	
E. R. Hendrickson	Air Pollution
Ralph Stone	Refuse
R. H. Bogan	Sewage
C. H. Hull	Water
M. A. Churchill	Stream Pollution
H. A. Faber	Public Health
Chairman, N. L. Nemerow	Industrial Wastes

has been prepared by the Industrial Wastes Section

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Paper 1612 April, 1958 SA 2

Journal of the
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ADVANCES IN SECONDARY PROCESSES OF SEWAGE TREATMENT IN THE PERIOD OCTOBER 1, 1954 TO JUNE 1, 1957

Progress Report of the Subcommittee on Secondary Treatment
of the Committee on Sewerage and Sewage Treatment
of the Sanitary Engineering Division
(Proc. Paper 1612)

This report of the Sub-committee Section III of the Committee of the Sanitary Engineering Division on Sewerage and Sewage Treatment summarizes from an engineering standpoint advances made in the secondary processes of sewage treatment during the period October 1, 1954 to June 1, 1957. The annual review[1,2,3] of the literature on sewage, waste treatment and water pollution by the Research Committee of the Federation of Sewage and Industrial Wastes Associations continues to summarize the progress made each year from the research, laboratory and operating standpoints.

New biological and physical methods of treatment are always under investigation. The endogenous respiration process, which reduces sludge and bacterial cell solids to a minimum, already has small-scale application for treatment of milk processing wastes. The microstrainer is[4,5] a promising device which has received some study for screening trickling filter effluents; performance has been reported as about comparable to that of final settling tanks.

Spray irrigation is among the new methods of secondary treatment growing in application. This is utilized most successfully for treatment of cannery wastes and, in some instances, for treatment of dairy and slaughter house wastes; it has not proved as satisfactory for treatment of citrus wastes in Florida.[6]

The re-use of treated sewage effluents, following secondary treatment processes, is an application continuously investigated. Among the latest developments in this direction is the re-use of treated sewage studied by the Los Angeles Flood Control District,[7] injecting effluent from the Hyperion treatment plant into recharge wells to force back saline water from the Pacific Ocean.

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Activated Sludge

Design

Most of the recent information with respect to design and operation of activated sludge units continues to consist of developments with so-called high-rate or short-period aeration. T. R. Haseltine has described a rational approach to the design of activated sludge plants.[8] He suggests that in conventional plants the loading should not exceed 50 lbs. of BOD per 100 lbs. of aeration tank solids and that moderate to large plants should be designed to operate at a Mohlman index of 250. The air required should be based on the ratio of BOD to aeration tank solids as well as on total BOD input.

Principles of activated sludge operation which influence design are described by W. M. Torpey and A. H. Chasick.[9] Design requirements for conventional activated sludge, step aeration, modified (high-rate) aeration and activated aeration are considered. These authors report that some changes affecting activated sludge treatment have been observed at the New York City Plants during recent years and raise the question, as have others, whether some element or type of detergent compound is adversely affecting biological sewage treatment. They point out that if the coaguability of the floc is reduced and its dispersion is increased, this might account for the observed changes.

Further information on the Kraus process developed in Peoria, Ill. has been made available.[10] In this process digested sludge is added to activated sludge and aerated in separate tanks. Advantages of the process are said to include: 1. ability to control the sludge settleability; 2. ability to treat industrial wastes and shock loads; 3. ability to maintain high suspended solids concentration in the aeration tanks; 4. ability to resist bulking and maintain stability in the activated sludge.

Kraus also describes the results of dual aeration wherein a part of the total air applied is admitted at the tank bottom through diffusers and part of the air is distributed at shallow submergence in a manner designed to create a maximum turbulent condition.

Additional information has been made available on the results obtained from the biosorption process at Austin, Texas, as reported by A. H. Ullrich[11,12] by comparison with the conventional activated sludge process at Austin. The biosorption process is said to require fewer adjustments, is not subject to upsets from shock loads or slugs of toxic materials, eliminates sludge bulking, reduces the froth control problem and requires less than one-half the aeration tank capacity. At Austin, the process is operated without preliminary sedimentation. During 1955, BOD in the raw sewage was 307 ppm and in the final effluent 20 ppm. The air supply was 1.59 cu. ft. per gallon. Ullrich suggests design data for a biosorption plant should take into consideration the BOD of the sewage to be treated and reports the design of the conversion of the Austin plant for influent with 200-250 ppm. BOD was predicated upon the following:

Q = average sewage flow
S = return sludge rate (Maximum = Q)

Sludge reaeration time, 2-1/2 hours based on Q. Mixing time, 20 to 35 minutes based on Q. Final settling tanks overflow rate, 700 to 900 gals. per sq. ft. per day based on Q. Air requirements 0.5 cu. ft. per gal. per 100 ppm of BOD to be removed in the biosorption process.

As a practical example of the application of the new developments in activated sludge, mention should be made of the new 47 mgd Miami,[13] Florida, secondary treatment plant. Local conditions at this site make it difficult to predict the exact degree of treatment which will be required, as winds and currents are involved in the offshore disposal of this effluent. Design and construction provisions have made it possible to operate this plant as conventional high-rate activated sludge treatment, as step aeration treatment, or as the new biosorption process. Another feature of this installation is the reclamation of effluent by lime softening and filtration for use as process and irrigation water throughout the plant.

Pasveer[14] described a simplified aeration process for small plants. The system consists of a large ring-shaped aeration basin with earthen walls in which raw sewage is aerated and recirculated by a rotating brush. The volume of the basin is about three times the dry weather flow. Sludge may be allowed to accumulate for as long as 2-4 months. The sewage is purified through the nitrification stage and the sludge may be dried on sand beds without any objectionable odor. Although the cost for air is about twice that of the conventional plant, the first costs and labor costs are very low. The land needed is about one square meter per capita.

Operation

It is well established that the amount of dissolved oxygen that can be carried in the conventional activated sludge plant depends not only upon the BOD and air inputs to the process, but also upon the amount of activated sludge solids carried. Although the conventional process may, or may not, utilize some form of sludge reaeration, the degree of oxidation of the sludge fed to the entering sewage must approach the endogenous state if danger of bulking is to be avoided. To accomplish this in municipal plants, it is generally agreed that (a) sufficient sludge solids must be carried to prevent the average BOD load exceeding 50 pounds per day per 100 pounds of sludge solids, and (b) some dissolved oxygen must be maintained throughout the aeration tanks. Perhaps most operators attempt to maintain a minimum of 1 to 2 ppm DO at the outlet of the aeration tanks, although some feel that a still higher DO is desirable. If sludge solids concentrations are increased to the point where BOD loads are less than 25 pounds per day per 100 pounds of solids, endogenous activity within the sludge requires so much oxygen that inordinate amounts of air are required to maintain the desired DO.

"Modified aeration" is not to be confused with "step aeration," "biosorption," or other processes utilizing sludge reaeration. It is defined as "a modification of the activated sludge process in which a shortened period of aeration is employed with reduced quantity of suspended solids in the mixed liquor." As a result of shorter aeration periods and lower sludge concentrations, less air is required and hence power consumption is less than the conventional process. However, the degree of purification is also less. Such plants generally have very limited blower capacity and generally operate at BOD loadings of 100 to 400 pounds per day per 100 pounds of sludge solids. Under such conditions the sludge never approaches the endogenous state and is always in an underaerated, oxogenous state. As such, it has a much lower absorption power than conventional activated sludge.

Operations of such plants are not agreed as to how much, if any, dissolved oxygen should be present in the aeration tanks. For example, at the Northeast

plant in Philadelphia,[15] the DO at the aeration tank outlets is measured every four hours and an attempt is made (not always successfully) to maintain 3 to 4 ppm DO. Since the BOD input is high and the air supply limited, the DO can only be increased by reducing the amount of return sludge. This increases the BOD to solids loading and hence may decrease the degree of purification obtained.

In contrast the operators of the Hyperion plant at Los Angeles[16] feel that the presence of any DO in the aerators is an indication of a waste of their limited air supply. They return sufficient sludge to avoid such a condition, but limit their sludge return so as to avoid the presence of negative oxidation reduction potential. They believe this gives them the minimum BOD to solids loading and hence the highest degree of purification obtainable with the existing facilities.

No doubt many activated sludge plant operators shudder at the thought of operating without any DO, but nevertheless the Los Angeles procedure may be logical. It would appear desirable to learn more about the adsorption power of activated sludges at various potentials from 0 to +300. If it varies with potential, then the problem may resolve itself into attempting to maintain some minimum ORP as well as control the BOD to solids loading. In this connection it should be pointed out that both Northeast and Hyperion are handling difficult sewages and have found their air supplies to be very limited. The air supplies at the New York City plants are not so limited and there it has been found feasible to maintain 1 to 2 ppm of DO while still varying the BOD to solids loading as required to maintain the degree of treatment deemed necessary.

Although it is common procedure to return digester supernatant to the head of the plant, such practice has caused difficulties at many small activated sludge plants, including the Long Road plant in Penn Township, Allegheny County, Pennsylvania.[17] That plant is a diffused air activated sludge plant of conventional design with two floating cover heated digesters operated in series. It was designed to serve a population of 13,800, but is presently serving something less than half that many people. Initially an effort was made to withdraw from 3,000 to 5,000 gallons daily of second stage supernatant at a uniform rate over the period from midnight to 6:00 A.M. and return it to the raw sewage wet well ahead of the plant. The rate of withdrawal was regulated by throttling a valve and constant attention was required to prevent its clogging. Withdrawals at higher rates produced a shock load that caused loss of DO in the aeration tanks and final clarifiers.

In June 1955 an existing tank 20' x 10' x 10' deep was equipped with air diffusion tubes so that it might be used for supernatant aeration. Although this eliminated the valve throttling problem and permitted less frequent withdrawals of larger amounts of supernatant, there was considerable frothing and even prolonged aeration did not greatly improve the settling quality of the black supernatant. However, it was found that the addition of an equal volume of return sludge to the supernatant and aeration of the mixture for 4 to 12 hours completely eliminated frothing and produced a rapidly settling brown sludge with a clear supernatant. The discharge of as much as 12,000 gallons of this aerated mixture to the incoming sewage over an hour's time not only had no deleterious effect, but actually resulted in increased DO and better settling activated sludge. To take full advantage of this unexpected dividend, operating schedules have been altered so that the aerated mixture is added to the sewage during the hours of peak flow and strength of sewage, rather than minimum. All around improvement in plant performance has resulted.

Modified aeration or high rate activated sludge plants in the United States have usually been operated with low solids and a detention period of about two hours. Wuhrmann[18] in Zurich has successfully operated pilot plants using relatively high concentrations of solids and very short aeration periods for several years. Popel[19] also reported that best results are obtained with high concentration of aeration solids and short aeration periods. Since March 1956, the Owls Head high-rate activated sludge plant in New York City has been operated at an accelerated rate with an average aeration period of one-half hour, at the same time producing results only slightly inferior to those attained during the previous four-year period, when the average aeration period was 2.1 hours, and using only about 50 per cent as much air. These results were obtained by increasing the solids in the mixed liquor and maintaining about the same sludge age which for four years of high-rate operation averaged 0.22 days.

Aeration

Since 1954, a number of articles pertaining to research and experimental work on improving the efficiency of aeration have been published. Tests have been reported in which oxygen absorption rates in aeration tanks have been determined by measurements in sulfite solution and deaerated water. This has been true for both the diffused-air and the mechanical type of aeration.

In the treatment of sewage, it is possible under certain conditions to obtain much higher rates of oxygen absorption than normally occur in municipal plants at the present time. Pasveer[20] in laboratory tests, using cotton filtered sterilized sewage and intense aeration of the diffused-air type obtained good purification at extremely high BOD loadings. Kalinske[21] reports that a mechanical air disperser, in which compressed air is injected into an impeller at the bottom of a circular tank, operated successfully at an unusually high BOD loading when treating a vegetable and fruit canning waste. Schulze[22,23] used a "Cavitator" unit, in which air is drawn in through a rotating hollow shaft and down into several rotor arms, for treating cheese wastes and pea canning wastes at high rates. This unit[24] was also operated on municipal sewage in a five-week test during the winter at a high BOD loading per unit of tank volume. However, there has been nothing published which would definitely indicate that municipal sewage can be continuously treated, satisfactorily and economically, in the conventional activated sludge process, at aeration tank loadings appreciably higher than in general use at the present time.

The relationship of the various factors affecting the rate of oxygen absorption in diffused-air spiral flow aeration tanks is shown by formulas developed by King[25,26,27] which indicate that the smaller the air bubbles and the longer they are in contact with the liquid before emerging at the surface, the greater the oxygen absorption per unit of power required; also, that the major part of the oxygen absorption is from the air bubbles with a minor part taking place at the liquid surface. Therefore, in a spiral flow aeration tank utilizing porous diffusers, the air should be diffused over as wide a band as is economically possible without reducing the overturning velocities below the intensity required for adequate mixing. Pasveer[28] arrived at a similar conclusion as the result of tests made with diffused air in small tanks in the laboratory.

A modification of diffused-air aeration, developed at Peoria, Illinois, is

described by Kraus.[10] The original plant had porous plate diffusers located adjacent to one wall in each of the aeration tanks. A second aeration system was installed supplying air to perforated pipe diffusers located in each tank about one-third depth, immediately adjacent to the wall either above the diffuser plates or on the opposite side of the tank. The latter position was the more efficient. Just sufficient air is diffused from this secondary system to cause a violent wave action in the tank.

Advantages claimed for dual air distribution include savings in power and increased oxygen absorption. When combined with the Kraus process for sludge index control in a nitrifying aeration tank, it is claimed that dual aeration results in the ability to effectively treat high BOD loadings using low aeration periods and low amounts of air.

Considerable research work has been done pertaining to the transfer of oxygen from air to water and sewage. Pasveer[29] discusses the theory of oxygen absorption from air bubbles ascending in water and presents experimental data on bubble sizes, rising velocities and the corresponding rates of oxygen absorption.

From experiments with brush aeration, he[30] found that by increasing the rate of oxygen addition the resulting greater turbulence reduces the size of the floc, increases its surface area and in this way increases the rate of diffusion of oxygen into the sludge. In a small tank, Pasveer found that the dispersed floc rapidly agglomerates to its normal size as it passes from the aeration tank to the secondary settling tank. He believes that by increasing the rate of oxygen addition and the turbulence, much more sewage can be purified per unit of tank volume than with conventional aeration. Wuhrmann has confirmed the importance of turbulence in laboratory experiments in Zurich.

Synthetic detergents in water reduce the absorption rates. Research work in regard to the effects of synthetic detergents in water has been done by Holroyd and Parker[31] and Lynch and Sawyer.[32] It is pointed out that the effects the detergents have on absorption rates in water should not be considered as applicable to the activated sludge process. King[26] found that the addition of activated sludge to sewage in an aeration tank raised to considerable extent the rate at which oxygen absorption took place. Eckenfelder, Raymond and Lauria[33] present preliminary results in which oxygen absorption in waste and waste constituents under standardized test conditions is related to water. The results indicate the variable effect of wastes on the oxygen absorption rate; also, that the rate approaches that of oxygen absorption in water during bio-oxidation.

Promoters of aerators of the type in which air bubbles are produced by mechanical means tend to emphasize that in this type of aerator a higher percentage of oxygen is utilized from the air than occurs with diffused-air aeration. However, it is not the oxygen absorption per unit of air applied but rather the power required per unit of oxygen absorption or unit of BOD removal that should be the basis for comparing the efficiency of the various types of aeration. Chain, et al[34] in studying the efficiency of different air spargers, using sodium sulfite solution in the presence of copper ions, found the porous sparger always more efficient than the perforated ring sparger. It was three times as efficient at the lowest agitation speed tested while with higher agitation speeds the difference became less pronounced but was still significant. According to Mohle,[35] the new high rate activated sludge plant at Wuppertal-Buchenhofen (Germany) has distributed the air in the aerator through open pipes arranged in a tapered pattern from influent to the effluent end of the tank.

At Manchester, England[36,37] plant size comparisons of air diffusion and mechanical aeration have been under test since 1935 at the Davyhulme Works. The three systems under comparative test are air diffusion; bio-aeration (Haworth) in which longitudinal circulation in channels is produced by means of paddle wheels; and Simplex surface aeration which is reported[38] to be under consideration for use in the proposed treatment plant extensions at London, England, to treat the sewage of a population of 1,600,000. Until March 1954 air diffusion showed up most favorably in the comparative tests from the standpoint of power and tank volume requirements. However, for the year ending March 1955, the two systems of mechanical aeration had improved appreciably while air diffusion system had deteriorated. The Simplex system was the most efficient and air diffusion the least efficient. Finch[39] in discussing these comparative tests, attributed the improved efficiencies of the Simplex and bio-aeration systems to improvements in the aerator mechanisms. The loss in efficiency of the air diffusion was caused by the clogging of the porous dome type diffusers. In contrast with the experience in diffuser clogging at the Davyhulme plant, King in a discussion[21] points out that at the West-Southwest plant of the Metropolitan Sanitary District of Greater Chicago, the clogging of porous plate diffusers has practically been eliminated. This has been done by double filtration of the air which first passes through an asbestos-lined cotton bag filter and then through an electronic filter. The filtered air contains less than 0.05 mg of dust per 1000 cu. ft.

A type of cast sparger first introduced in 1953 does not require a clean air supply. Diffusers of this type have replaced the original porous diffusers at the Hyperion Plant in Los Angeles and the Northeast plant in Philadelphia. Eckenfelder has measured the oxygen transfer efficiency of such spargers, saran-wrapped tubes and porous plates at full scale installations in the San Antonio plant. His work indicates that all three types may have about equal efficiencies when operated at their optimum rates. However, those optimum rates differ considerably for the different types of diffuser.

Air consumption per pound of BOD removed has been about 60 per cent higher at the Hyperion plant in Los Angeles than in Chicago and New York City. This excessive air consumption is attributed to long flow time through the trunk sewer and the high annual temperature which cause the sewage to arrive at the plant in a highly septic condition.

Trickling Filters

New research continues to provide new developments and applications in secondary processes. Trickling filters may be affected by the development of molded polystyrene plastic media.[40] This material is reported to exhibit several structural advantages and may be able to produce a better effluent than conventional media. Research is also under way to determine the suitability of precast concrete channels,[41] placed horizontally and filled with coarse filter material.

Design

The efficiency of a trickling filter is appreciably affected by the rotating speed of the distributor arms. Tomlinson and Hall[42] reported from studies with a four arm distributor at Minworth, England that the optimum rotational speed was in the range of 15 to 30 minutes for each revolution. Surface

ponding which occurred regularly with a jet driven distributor at 1 to 5 minutes per revolution, did not occur when the speed was reduced. The efficiency of the filters with the slower distributor was more constant and at a higher level than with the jet driven distributor.

In the usual high rate trickling filter, recirculation of effluent is required to obtain sufficient hydraulic loading so that sludge is continuously removed and the slime layer kept thin and active. If the filter is deep and the sewage flow sufficiently high, the organic loading can be greatly increased and the filter slime kept under control. Imhoff[43] and Schulz[44] reported that an experimental filter, 26.4 feet deep and 3.28 feet in diameter, was operated at organic loadings of 5.2 to 6.5 lbs. BOD per cu. yd. per day and a hydraulic loading of about 120 m.g.a.d. The BOD removal was about 90 per cent at 5.9 lbs. BOD per cu. yd. per day. Independently, Ingram,[45] in a filter 18 feet deep and one foot in diameter, applied the principle of incremental feeding and the addition of air at several depths. The filter was constructed with six three-foot sections of transite pipe in such a way that sewage could be added to each of the top four sections and air to each section. When the sewage was added to the top three sections at a rate of about 120 m.g.a.d. and an organic loading of 8.57 lbs. BOD per cubic yard per day, he obtained 88 per cent removal of BOD. A strong flow of sewage through the filter is essential.

Rampf[46] explains the theory of filter efficiency as a function of contact time between sewage and the biotic culture of trickling filter. Differing from American practice, high rate trickling filters in Germany are designed with deep beds (10 - 12 feet), and little recirculation (0 - 0.3:1 for domestic sewage). A contact time is obtained that is equivalent to that obtained by recirculating flow through shallow filters. Precast block underdrains with 40 per cent free space, a peripheral effluent channel, high walls, and uniform aggregate afford adequate aeration of these filters. Loadings of 20 m.g.a.d. of sewage with a BOD of 157 ppm results in 91 per cent reduction at the Frankfort Airport. A minimum dosing of about 15 m.g.a.d. and intermittent sprinkling of five minute on and off cycles are also current German design trends.

A review of design loadings used for high rate trickling filters has been presented by J. A. Montgomery.[47] Current design trends include deeper filters (6 to 8 feet deep) which are well aerated and lower dosing rates (800 - 2000 pounds of BOD/Acre Foot/Day). These represent changes from design practice, compared to several years ago, of high rate trickling filters which were 3 and 4 feet deep with loadings of about 3000 pounds of BOD/Acre Foot/Day. These design trends are due in part to an increased number of states which now use certain design standards such as the Tentative Standards of the Upper Mississippi and Great Lakes States as guides for review and approval of plans for municipal sewage treatment plants.

J. H. Sorrels and P. J. A. Zeller[48] have studied the performance of two stage trickling filters and conclude that intermediate settling is not necessary as no excess sludge forms which would cause clogging and ponding of the second filter.

Developments are also on the large size: the installation of Biofilters at El Paso,[13] Texas, four 220-ft. diameter units, is believed to constitute the world's largest.

With the use of an experimental trickling filter using vertically suspended wire screens as a carrier material for the biological film, Schulze[49] found that the maximum thickness of the film is 2 or 3 mm. if anaerobic zones are

to be avoided. This explains the need for high hydraulic loading with high organic loads.

Operation

It is generally agreed that recycling of filter or final settling tank effluent through a high rate filter improves its efficiency and recycle ratios of 1 to 3 are common with ratios as high as 10 being occasionally used. However, there is considerable disagreement among designers as to whether or not the BOD of the recycle should be included in calculating the organic loading on the filter. Some ignore it completely and those that include it do not distinguish between its "treatability" and that of the incoming raw or settled waste. While such differences may be of little significance where the filter is treating normal municipal sewage, experience at the Springboro,[50,51] Pennsylvania plant of the Albro Packing Co. indicates quite the contrary for a roughing filter handling a concentrated waste. At that plant a single stage filter 47 feet in diameter with an 8.5 ft. media depth is handling screened and limed pickle and kraut wastes having a BOD of 800 to 3,000 ppm (average for 1955 was 2,020 ppm). Hydraulic loading ranged from 7.9 to 10.3 and averaged 8.75 m.g.a.d. The recirculation ratio ranged from 1.66 to 7.07 and averaged 3.2. Organic loadings, excluding recycle BOD, ranged 356 to 2,820 and averaged 1,585 pounds of BOD per acre foot per 9 hour day of maximum waste flow. (On the basis of a 24 hour day, the loading was probably about 50 per cent greater.) The BOD recycled to the filter ranged from 35 to 75 per cent of the total BOD applied to the filter, and averaged 60 per cent, that is the total organic loading including recycle averaged about 4,000 pounds of BOD per acre foot per 9 hour day. It ranged from 3,000 to 10,500. The BOD removals through the filter and final settling lagoons varied greatly and ranged from 36 to 91 per cent. The average was only 71.7 per cent as compared with an expected and required removal of 85 per cent. A study of the data failed to show any correlation whatsoever between the percentage BOD reduction and either the recirculation ratio or the organic loading. This was true regardless of whether the organic loadings included or excluded the BOD of the recycle. However, further study clearly showed a definite relationship between the per cent BOD reduction on the one hand, and the per cent of recycle BOD in the total BOD applied to the filter. So long as the recycle BOD ranged between 35 and 65 per cent of the total, the relationship seemed to follow a straight line with the BOD reduction ranging from 91 to 75 per cent respectively. However, if the recycle BOD accounted for more than 70 per cent of the total, the efficiency of the filter fell off very rapidly, i.e., a BOD reduction of only 36 per cent was obtained when 75 per cent of the total organic loading consisted of recycle BOD. As a result of this study, it was decided to reduce the recirculation ratio and hence the amount of BOD recycled. It was believed that if the recycle BOD could be limited to about 40 per cent of the total BOD loading, the filter and lagoon would consistently effect at least an 85 per cent reduction in the BOD of the raw waste.

Sewage Stabilization Ponds

Sewage stabilization ponds provide a form of secondary treatment of perhaps the widest present research interest. For communities located in warm, dry climates, having adequate area available, they have long given good

results. Their use has not been confined to these warm, dry areas but they have spread particularly through the Northern Plain States. In 1956 there were 41 such facilities in operation in North Dakota, 32 in South Dakota and 136 in the Missouri River drainage basin.

In these ponds, bacteria aerobically attack the organic matter of the sewage and convert it to more stable forms. They also liberate nutrients, particularly carbon dioxide, which are necessary for algal growth. The algae, with the aid of light, utilize the carbon dioxide and liberate oxygen to maintain aerobic conditions. During the long period of ice cover, photosynthesis stops because of lack of light and aerobic processes are replaced by anaerobic ones. The ice layer prevents odors from escaping from the pond. The critical period occurs during the transition from ice cover to open water. Field tests by the USPHS[52] have indicated that, at loadings of 100 persons per acre, the recovery in the spring will require a minimum number of days. At other seasons aerobic conditions can be maintained with much heavier surface loadings. Results from five installations showed BOD reductions generally running from about 70 per cent in winter to about 90 per cent in summer.

The use of oxidation ponds is emerging and fast developing into several distinct treatment processes. W. J. Oswald and H. B. Gotaas have described the three distinct uses and two types of sewage oxidation ponds.[53] These uses are: 1. Holding reservoirs for partially treated effluents; 2. Leaching reservoirs for percolating liquid wates; and 3. Primary or secondary waste treatment. The two types of treatment ponds are differentiated with respect to the method of acquiring oxygen: Type I which is dependent upon surface aeration and Type II in which the oxygen is dependent almost entirely upon algal photosynthesis. Examples of Type I ponds are becoming increasingly popular in the Missouri Basin States while ponds strictly of the Type II characteristics have only been developed experimentally at the University of California. The large majority of the oxidation ponds currently in use or under construction in California and Texas are intermediate between Types I and II as explained by I. Nusbaum.[54] The development of algal oxidation ponds may offer a compact and economical treatment method in locations with favorable climatic conditions.

Practical design considerations for Type II pond for use in municipal sewage treatment are: a. Detention time—varies from 1 day in summer to 6 days in winter, optimum 3.5 days; b. Depth—varies with light and strength of sewage, optimum 18-inches; c. Reduce deposition of sludge by multiple inlets, small depth, dilution by recirculation, orientation of pond for mixing by wind and wave action, and vertical mixing; d. Recirculation to supply oxygen to bacteria consuming the organic material in the sewage; e. Separation of algae from treated sewage by sedimentation or screening; f. Paving to prevent aquatic growth and reduce turbidity; and g. BOD loadings in summer of 225-250 pounds/acre/day and winter of 100-125 pounds/acre/day.[55]

Results of pilot plant studies are quite promising in that 95-97 per cent of the BOD of sewage was reduced in one week's detention time.[56] In addition, a system of continuous precipitation of algae was developed by which one ton of dry algae was recovered for each million gallons of sewage. The dry algae contain over 45 to 55 per cent protein and may be worth as much as $100.00 a ton for animal food, which would quickly amortize construction and operating costs.

E. F. Gloyna and E. R. Hermann[57] have commented upon design considerations for oxidation ponds and included an evaluation of these factors for both

pilot and full scale plant operations in Texas.[55] Primary considerations for design include an evaluation of intensity and duration of light; temperatures of air, ground, and waste; evaporation and percolation rates, rainfall, soil stability conditions, and disposal or use of effluent. Loading factors for Central Texas of 200 pounds BOD/acre/day, 150 pounds BOD/acre/day, and 50 pounds BOD/acre/day for ponds of which the effluent is to be used for irrigation water, disposed in adequate streams, or intermittent streams respectively are presented. Recirculation up to 1:1 may be required for unsettled strong wastes. The depth of the first third of the pond area should not be over four feet. The use of series ponds is recommended as algae will settle in the secondary ponds which will produce a clear effluent. The above criteria refer to ponds intermediate between Types I and II and are of a type commonly in use in the Southwest.

The use of ponds for the treatment of raw sewage has become popular in many small communities in the Upper Missouri Basin area as reported by J. K. Neel and G. J. Hopkins.[58,59] They report that design practice in North Dakota has been to use a 10 acre pond about 4 feet deep with about 200 days detention for a community of 1000 persons. In the first stages of development of these ponds in about 1948, nearly all were designed for virtual detention. Now most of the new ponds are designed for overflow and improved circulation. The largest pond of this type is at Jamestown, North Dakota, where a 130 acre pond serves 10,000 people.

A series of interesting experiments were conducted at Kearny, Nebraska[58] to determine if a 10 acre pond, 3-3/4 feet deep could effectively treat the raw sewage from a city of an equivalent population of 12,725. This pond functioned in an aerobic manner in the summer with prolific growths of green algae and anaerobicly under ice cover in the winter. However, odors did develop in the spring and fall when anaerobic conditions persisted with no ice cover. It was concluded that the pond did effectively treat the sewage with the exception of the odor nuisance several weeks of the year (90 per cent BOD reduction in summer, 32 per cent in winter), photosynthesis was the major source of oxygen, settleable solids are completely removed, 95 per cent of the coliform organisms are reduced, and the bottom should be impermeabilized to stop ground water pollution.

This experience indicates that the existing design criteria with respect to organic loading in use in these states may be too conservative. Other design factors in current use in these states are riprap bank protection for light soils, no reduction in overall volume due to sludge deposits, dikes should be protected by seeding and fencing, series ponds used for more effective treatment, and greater depth for larger ponds (7 to 8 feet for 20 acre ponds).

The use of oxidation ponds offers a means of economical secondary treatment in regions of favorable climate and low land values as explained by M. B. Allen.[60] Average construction costs in South Dakota of $16.28/capita have been reported.[58] It is anticipated that the use of the various types of oxidation ponds will continue to flourish not only for municipal treatment but in addition for industrial wastes and in particular canneries.

Removal of Radioactivity

Research developments demonstrate that micro-organisms can concentrate certain types of radioactivity from sewage and wastes. Dobbins, Edwards and

Gulevich[61] studied one- and two-stage trickling filters for laundry wastes containing fission products and other isotopes. They found that in wastes having gross beta activity in the range of 800 to 1100 d.p.m. per ml. of 200 day old fission products or in the order of 10 d.p.m. per ml. of specific isotopes, percentage removals were Sr, 69; Ru, 79; Zr, 79; Y, 87; Ce, 97 and fission products 90, under optimum conditions.

Experiments on the use of oxidation ponds for removing radioactivity by Steel and Gloyna[62,63] showed that 20 to 25 per cent of the activity was removed for each 5 days' detention in the pond.

Dobbins[64] concluded that biological methods have a limited application for the removal of radioactivity from liquids. The extent of the removal depends upon the biochemical relationship which exists between the element and the biological system as well as the chemical composition of the waste. For elements in cationic form, the removal will be greatest for those having high valence states. Probably the best application of biological methods is for the treatment of low activity level laundry wastes which contain complexing agents which interfere with chemical treatment methods.

Effect of Synthetic Detergents

An excellent report on the effects of detergents on sewage treatment was published by the Ministry of Housing and Local Government[65] of Great Britain. The available information on the problems caused by detergents, especially in activated sludge plants, was compiled and evaluated.

Raybould and Thompson[66] in large scale experiments found that the detergents Tide and Santomerse had no ill effect on trickling filters or Haworth-type activated sludge units if the plants had sufficient capacity and were in good condition.

Respectfully submitted,

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A STUDY OF SEWAGE COLLECTION AND DISPOSAL IN FRINGE AREAS

Progress Report of the Committee on Public Health Activities of the Sanitary Engineering Division
(Proc. Paper 1613)

INTRODUCTION

The accelerated and uncontrolled growth of the urban fringe around American cities during the last three decades has created serious problems in providing essential community facilities and services. Finding a way to provide these services is a major challenge to many concerned professional groups as well as to the public. It is the broad purpose of this report to indicate the role which the engineering profession, and particularly those individual engineers and planners concerned with urban development should assume in planning for and providing one major service: Sewage collection and disposal.

Providing adequate sewerage and sewage disposal facilities in fringe areas as a part of orderly community development requires a high degree of engineering skill, not only in the conventional technical sense but in organization, financing, and planning, all of which must be closely coordinated with other professional groups. The planner, the economist and financier, and the political scientist, as well as the engineer, have important responsibilities in the development of sound metropolitan area plans. The engineer has a special responsibility for developing and designing sewage works components that are consistent with the master plan.

Within the engineering profession there is need for close cooperation between engineering groups employed in government and those engaged in private practice if orderly, planned growth of sewage works in fringe areas is to be attained. Each group has a vital contribution to make to the effort.

Engineers employed in local governmental agencies responsible for building and operating sewage works in fringe areas can be helpful in seeing that the administrative patterns are kept broad and comprehensive and the responsibility for sewage services is not split among a multiplicity of small independent agencies. These engineers can give the private engineering consultant much helpful support for regional planning of sewage works.

The engineers of state and regional health and water pollution control

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agencies with responsibilities for establishing broad policies that protect public health and welfare can play a vital role in stimulating local action toward acceptance of their governmental programs.

Engineers of the Public Health Service and the Housing & Home Finance Agency have made major contributions in evaluating the basic factors requisite to successful installation and operation of individual sewage disposal systems. In cooperation with state health departments, these studies need to be extended to include (1) the design, economy, and efficiency of various types of sewage disposal systems for small developments, (2) interim facilities to be replaced in accordance with master plan as growth and development occur, and (3) stage construction of large treatment plants.

In solving the fringe area sewage works problem the consulting engineer in private practice can be a major leader in developing integrated plans which transcend local and even state political boundary barriers. He is in a particularly favorable position to study the problem from the broad, over-all viewpoint and to assist in resolving the difficulties normally encountered when several political entities are involved.

Engineers in the employ of official national organizations are in a position to provide assistance and leadership through the development of policies, standards and requirements of lending agencies. Adoption of adequate standards by these agencies will give inestimable support to proper provision for sewage works in fringe areas supporting programs directed toward proper solution to the sewerage and sewage disposal problems of fringe areas.

The engineering educator has a responsibility to bring to the attention of civil engineering students the complex nature of this problem and the application of engineering principles to its solution. He also has the opportunity to stimulate and conduct research.

The engineer in the local or regional health department is a key man in the actual execution of all phases of any integrated, long-range sewage works plan. Because of his knowledge of local problems and attitudes, he can play an important part in securing needed approval of funds for the engineering study, and in assisting in certain technical phases of that study. He can advise metropolitan area planners, plumbing and building regulatory officials, subdividers, and builders on locations and conditions suitable for various types of sewage disposal systems. He is in a particularly favorable position to review proposed designs and installations because of local experience with operation of such systems and devices. Finally, he has an official interest in the best solution since he necessarily continues to be associated with the system long after it has been completed and placed in use.

The specific objective of this initial report is to provide the engineer with further information on (1) the nature and scope of suburbanization and its impact on the design and operation of sewage works, (2) the major needs in planning, designing, financing, and administering sewage works in fringe areas, (3) effective administrative devices now being used to plan, finance, and operate systems in suburban areas, and (4) the role of the engineer in solving urban fringe problems. It is hoped that the report will stimulate the engineering profession to more active, cooperative joint effort toward a solution of the problem.

Suburbanization

Suburbanization refers to the blending of urban and rural life. This process

is dominating the development of cities and fringe areas. Although suburbanization is present in all cities, for the purpose of this study, the problems of the standard metropolitan areas* will be considered.

According to the 1950 census there are 168 SMA's* in the United States. In these 168 SMA's, embracing 7% of the land area, live 57% (85,000,000) of the nation's population. They range in size from metropolitan New York with a population of 12,912,000 to Laredo, Texas, with a population of 56,000. The typical metropolitan area structure includes a central city and one or more satellite cities and an urban fringe which has scattered developments penetrating at various depths into the rural areas, as shown in Fig. 1. Their distribution throughout the United States is shown in Fig. 2. Between 1940 and 1950, the population of the central cities of the SMA's increased 13.9%, while that of the fringe areas increased 34.9%.

Further study of the census data shows that the differences in population growth have qualitative aspects. For example, the increase in the central city population included the lowest income group, the aged, and the physically handicapped. The increase in the new fringe area was largely the medium to high income groups, the young and vigorous.

The 1950 census showed 16,120 units of government and 2,898 special districts in the Nation's 168 SMA's (Figs. 3 and 4); consequently, the problems of sound economic development of sewage works are compounded by the bewildering overlap of special districts in these areas.

The disconnected and noncontiguous growth of the new urban fringe areas around most American cities is primarily responsible for much of the difficulty encountered in providing an orderly transition from the individual septic tank system to that of public sewers. This problem would seldom occur if land subdivision were rigidly controlled by the central city, as in Europe, where the land immediately adjacent to the central city must be developed before more distant areas. The pattern of development in Wyandotte County, Kansas, is typical of suburban growth in America (Fig. 5). It is evident that provision of sewers and sewage treatment for the widely scattered subdivisions and communities would impose an excessive cost on them if the initial facilities were sized to serve the area when fully developed. While the problem is most acute in the larger metropolitan areas, it is not entirely limited to them. As a matter of fact, the solution to the fringe area sewage works problem is sometimes more difficult in areas surrounding smaller communities with populations of less than 50,000.

The practicing civil engineer is therefore faced with the need for developing a series of small sewer systems that can be economically integrated into

*The term standard metropolitan area (SMA) was developed by the Bureau of the Census in an effort to provide fixed areas that would be useful in the study of urban and rural trends. The SMA is defined as an area containing a population in excess of 50,000 persons and a population density greater than 150 persons per square mile. The boundaries of the metropolitan area are the boundaries of certain counties or groups of counties. The term "standard metropolitan area" replaces the term "metropolitan district" which has been used by the Bureau of the Census since 1910. Because of the nature of its definition the outer boundaries of the metropolitan district changed considerably in each decennial Census Report. This has been largely overcome by the new SMA definition. For further information see Population Series A1 1950 Census Reports.

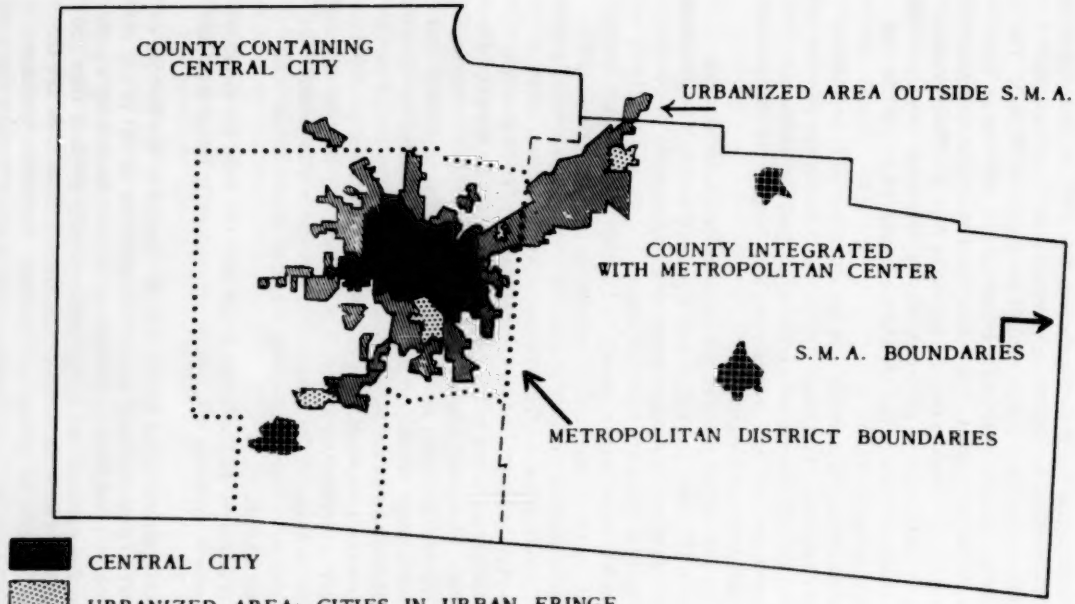
COUNTY CONTAINING CENTRAL CITY
URBANIZED AREA OUTSIDE S. M. A.
COUNTY INTEGRATED WITH METROPOLITAN CENTER
S. M. A. BOUNDARIES
METROPOLITAN DISTRICT BOUNDARIES
CENTRAL CITY
URBANIZED AREA: CITIES IN URBAN FRINGE
URBANIZED AREA: BALANCE OF URBAN FRINGE
OUTLYING URBAN PLACES IN S. M. A. RING (INCORPORATED OR UNINCORPORATED)
RURAL-FARM AND RURAL-NONFARM TERRITORY IN S. M. A. RING

FIGURE 1 – SCHEMATIC DIAGRAM SHOWING TYPES OF URBAN AREAS

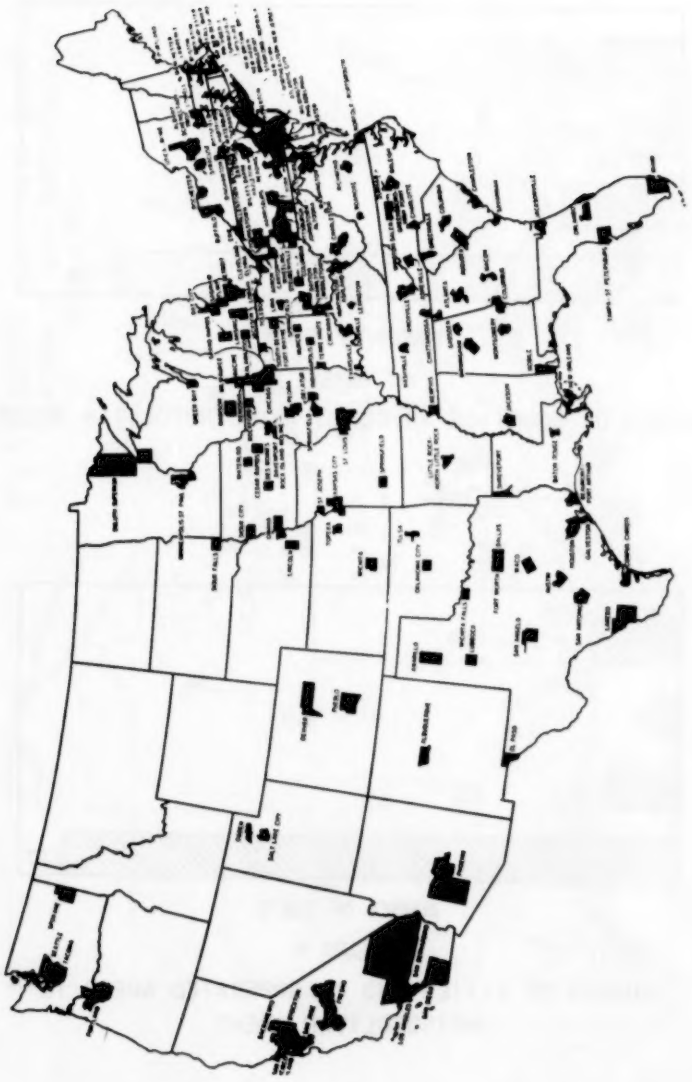

FIGURE 2 – STANDARD METROPOLITAN AREAS: 1950

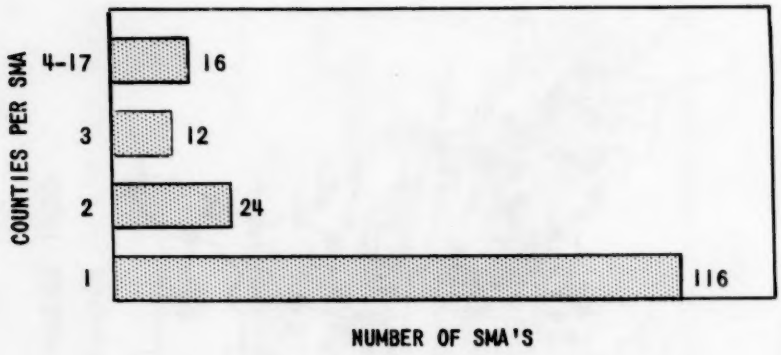

FIGURE 3

NUMBER OF COUNTIES INVOLVED IN METROPOLITAN AREAS

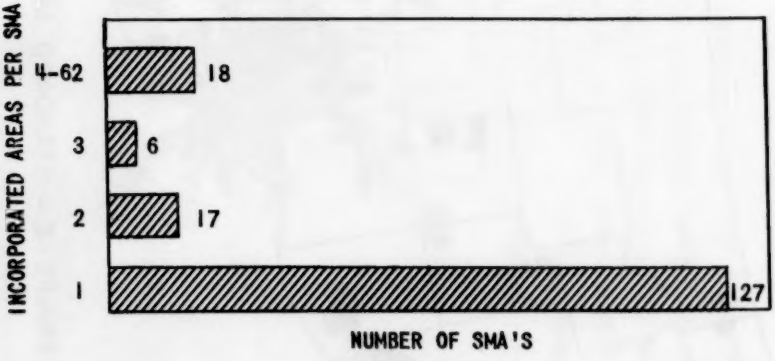

FIGURE 4

NUMBER OF CITIES AND INCORPORATED AREAS IN METROPOLITAN AREAS

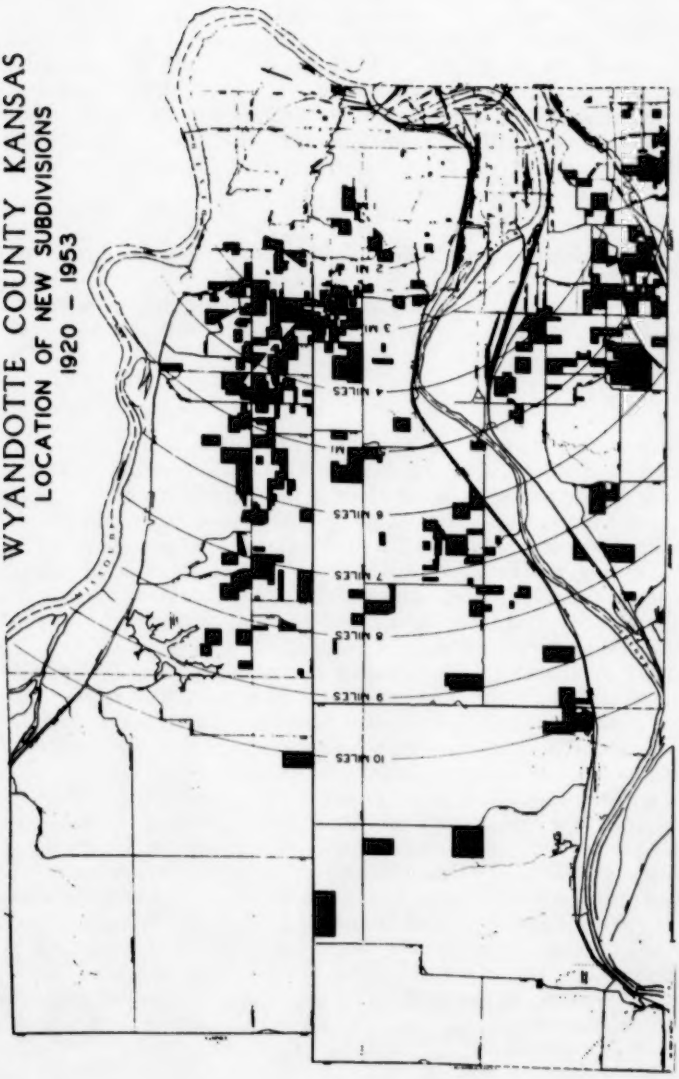

FIGURE 5 – DEVELOPMENT IN WYANDOTTE COUNTY, KANSAS

a comprehensive system as the sparsely populated fringe area approaches full development. The planning and development of solutions to the problem involve a maze of legal, financial, and administrative problems that frequently make the problems of engineering design appear small by comparison.

Long-range planning on a metropolitan basis is obviously a desirable prerequisite for solving the problem. The finding of acceptable administrative mechanisms to do the sound planning and design and to provide equitable sharing of the financial, legal, and administrative burdens appear to be a major undertaking in most metropolitan areas. No single mechanism appears to be acceptable to all communities. To date, a great variety of administrative patterns are being used.

Major Needs in Providing Adequate Sewage Disposal Systems in Fringe Areas

The problems encountered in developing sewerage systems in fringe areas can be grouped into four general categories: Planning, financing, administration, and design.

Planning

In the planning category the most serious need is for a master plan for sewage works as an integral part of a comprehensive metropolitan area development plan. Few area or regional plans developed by official metropolitan planning commissions currently include a master plan for sewerage and sewage disposal. Development of such a plan on a basis of sound economic data and comparative cost analysis should be adopted as standard practice. Proper planning and financial procedures would permit acquisition of the logical sites for treatment plants, permanent and interim, and would avoid needless delays in construction of essential facilities.

While prime responsibility for developing the master plan should rest with metropolitan planning agencies, they should utilize to the fullest degree, the skills, training, and services of consulting engineers. The latter in turn should work closely with official agencies in developing plans for smaller systems to assure that designs are consistent with the master plan.

The planning agency, utilizing sound engineering analyses of local conditions, may develop the most reasonable solutions to problems concerning—where and on what basis individual septic tank systems are to be permitted; where small subdivision systems with temporary treatment plants will be permitted; and where sewers must be extended and connected to the trunk sewers.

The need for sewers and treatment facilities as elements in the metropolitan area plan has already been stressed. After many years of attempting to solve its problem by the use of small independent systems, St. Louis, Mo., has developed a Metropolitan Sewer Plan. Similar progressive action has been taken in Nashville, Tennessee; Los Angeles, California; Miami, Florida; and many other metropolitan areas.

Financing

Present methods of public and private financing are reasonably adequate insofar as lateral sewers are concerned. The main fringe area problem is providing the necessary connecting and outlet facilities for the numerous lateral sewers associated with scattered developments.

Lending agencies generally do not give adequate recognition to the value of public sewers in their loan appraisal practice. Although septic tanks for urban type developments are usually inferior to public sewers in performance and convenience, lending agencies often fail to grant sufficient increases in loan value to induce the developer to provide sewers. Home purchasers do not fully appreciate the value of public sewers, and consequently exert insufficient demand for the community facility. Information on the benefits this system normally offers could be used to stimulate both the developer and the home owner to secure it.

The prime responsibility for implementing the master plan should rest with those local governmental agencies responsible for the construction and operation of the systems.

It is essential that procedures adopted be rigidly adhered to by all engineers concerned with implementation of the waste plan.

Past experience in metropolitan areas indicate fringe areas are seldom able to move directly from rural methods to an integrated metropolitan system but the transition is more likely to occur in steps passing from individual septic tank systems to subdivision systems, to small watershed systems, and finally to a metropolitan system. Nothing is inherently wrong with such an approach, if the planners, engineers, and most important of all, the property owner understand that these are interim measures.

The economic and social desirability of making the transition in as few steps as possible is obvious and the public agencies should be alert for opportunities to bypass one or more steps in the procedure.

State and local agencies responsible for controlling stream pollution and reviewing plans for sewage works need to re-examine their policies as they apply to the suburban problem and determine whether they produce the desired response from the general public—a steady progress towards coordinated sewer systems for an entire metropolitan area. The lowering of treatment and material standards for permanent plants is not suggested, but there is some merit in lowering these standards for interim plants located on temporary sites, provided this permits a reduction in investments which can be used in the extension of the trunk sewers to points nearer the permanent site.

It is necessary that both consulting engineers and engineers in health departments recognize that minimum design standards for permanent plants must be relaxed somewhat when applied to interim installations. Unless this is done it will seldom be economical to use interim plants.

Administration

The decentralized, widely scattered nature of fringe developments makes the use of interim community sewage treatment facilities necessary in many areas. Provision for successful operation and maintenance of such plants often require the establishment and use of special administrative agencies not hitherto required in sanitary engineering practice.

To solve effectively the problems of operation and maintenance of community plants and to insure their ultimate integration into a regional system, it is highly desirable, if not essential, that responsibility for planning, financing, and operating these small systems be vested in a public agency of some type with authority over all sewage works in a given watershed. Experience in St. Louis, Mo.; Los Angeles, Calif.; Seattle, Washington; Nashville, Tenn.; Miami, Florida; and numerous other areas has demonstrated the need for a

central control agency. Occasionally the central city can fill this need but usually a separate body must be created.

Equally as important as the centralization of responsibility is the provision for the employment of competent supervisory and operating personnel. Providing for this need is most difficult at small plants serving individual housing developments. Some metropolitan areas have solved the problem by creating a large sanitary district that supervises all the plants within the district. The large district can employ a competent engineer to supervise all the plants and associated collection systems and spread the costs among the several sewer districts receiving services. In the St. Louis, Mo., area, maintenance and operation was provided by a private company that contracted with the various sewer districts to operate and maintain the sewer systems and treatment plants. Such an arrangement is satisfactory if the contract is properly drawn and supervised. In some areas, arrangements have been worked out with nearby cities for their personnel to operate the interim plant. In most states funds for operation and maintenance purposes may be provided by tax levies or sewer service charges.

Design

Many engineers who design sewage works for fringe areas have found the work discouraging because they are not permitted to plan the work on a sound engineering and economic basis. Local political considerations, selfish developer interests, and the absence of a master plan for the entire area are the principal reasons for the difficulty. This situation has not been portrayed to the general public in such a manner that the individual property owners can see clearly the benefits that will accrue to them if master planning were practiced.

The economy of designing sewer systems on a topographical basis was clearly pointed out by McCallum[1] in 1936 in his discussion of a comprehensive sewerage plan for Nassau County, New York. He indicated that a potential saving of 14 million dollars in construction costs and 23-1/2 million dollars in operation costs, interest and amortization costs over a thirty-year period could be saved by developing 14 watershed systems instead of 65 to 70 small independent village systems. Since this estimate was made in 1936, the dollar savings would be even greater today.

Regardless of the difficulties, the engineer should continue, to the greatest degree possible, to employ the following practices in designing sewage works in fringe areas.

(a) Design and construct each portion of the system in conformity with the master plan whenever such a plan is available.
(b) Install the sewers prior to construction of houses. (It is suggested that, as a minimum, public sewers be required when the population density reaches 125-150 per square mile; that, as a goal, sewers be considered whenever the land is subdivided into tracts of less than 1 acre.)
(c) Keep capital investments in plants located on temporary sites at a minimum. This particular item needs much more study with regard to type of plant, location, construction materials, operation and maintenance, and sludge handling procedures.

The following uneconomical practices should be avoided:

(a) The construction of numerous small independent sewer systems in a

single drainage area.

(b) The design and construction of trunk sewers without making provision for serving the needs of the area when fully developed.
(c) Investment of excessive capital in treatment plants located on temporary sites.
(d) The installation of combined storm and sanitary sewers.
(e) The postponement of sewer construction until development of the area is well advanced.
(f) The use of individual septic tank systems in urban areas, particularly where the population density exceeds 125-150 persons per square mile.

Individual systems are usually preferable to community systems where (1) soil and ground water conditions are especially favorable for private sewage disposal systems, (2) state or local requirements provide for adequate and properly designed private systems and (3) subsurface disposal must be employed for lack of a suitable watercourse. They require larger lots.

The following types of treatment plants are most commonly used for small communities: Imhoff tanks and sand filters, package type activated sludge and trickling filter systems, waste stabilization ponds, and septic tanks with subsurface disposal. Any of these types may serve the purpose if properly located, designed, and maintained. It is highly desirable to provide a protective distance of at least 500-750 feet between the plant and the nearest housing to prevent odor nuisances. Greater isolation should be provided if possible. Experience has shown that sludge digestion and drying operations at small community plants are the most frequent cause of odor complaints; therefore at plants with minimal isolation the removal of sludge to more isolated plants for digestion and final disposal should be given careful consideration.

Organization Procedures in Metropolitan Areas

Correspondence with states, counties and cities; field studies of three selected metropolitan areas—Kansas City, Houston, and Seattle—and a review of the literature have revealed a variety of methods in use at present to provide sewerage service in fringe areas. These methods may be grouped into two general categories: Those privately financed and those publicly financed. Included in the first category are individual systems and privately built community systems; in the second category are cities, townships, and county sewer districts, conservancy districts, metropolitan districts and authorities, municipal federations, city and county consolidations, and multiple city-county contracts.

Privately Financed Systems

Individual Systems.—Because of its wide use, the septic tank is considered herein to represent individual systems. Other treatment processes producing a final effluent may be considered in this category. The role of the septic tank is subordinate to that of the disposal field, but for ease of reference, the method of on-lot disposal of sewage by soil absorption will be referred to as the septic tank system.

Under proper circumstances the septic tank system can be a satisfactory means for disposal of sewage in developing suburban areas. It should never be used unless favorable conditions exist. These conditions have been reported

in technical publications and are described in The Manual of Septic Tank Practice. (Department of Health, Education, and Welfare, Public Health Service.)(2) It is anticipated that general application of the recommendations contained in the manual will result in rejection of the septic tank system where conditions are not proper for its successful operation. These conditions can be summarized briefly under soil, design, construction, and use.

Of these factors, soil structure is the most important and it is the one over which the engineer has the least control. To be suitable for subsurface disposal of sewage there must be an adequate cover of acceptable soil between the bottom of the disposal field and rock or other restraining formations; the ground water table must be well below the surface of ground; and the soil must have an acceptable sustained percolation rate. Some of these requirements can be reduced for isolated rural dwellings, but they should be strictly adhered to in suburban areas. Unfortunately, the soil in over half of the United States fails to meet these requirements. Furthermore, unsuitable soil is likely to be found in high ground areas desirable for house building, while suitable soil is likely to be found in low land areas desirable for agriculture. Since no practical method has been developed for improving the soil structure, the wise builder will make a preliminary investigation before purchasing a tract of land for housing development. Soil maps coupled with an honest inclusion of operational experience with existing systems in the area are very valuable aids in evaluating the suitability of the soil. In some localities soil maps are used to show "critical areas" where soil, topography, ground water or other conditions preclude successful septic tank operations. The engineering profession should encourage the development of these maps in all rapidly expanding areas.

Adequate capacities in all parts of the system is basic to the design. With increasing water consumption and modern appliances, the need for increased tank capacity has been readily accepted. Surprisingly, there has been resistance or at least apathy to expanding the absorption field even though this portion of the system is directly concerned with the disposal of the daily volume of liquids. The relationship between soil area and the life of the system is difficult to convey, but nevertheless, it exists. Each year an increment of the available area becomes impervious and noneffective. When the remaining area can no longer absorb the daily volume of liquid the system fails and partially treated sewage appears on the surface of the ground. Therefore, a large absorption area over and above that needed to dispose of the daily volume of liquid should be provided to insure a long period of trouble-free operation.

Protection of the soil during construction is important. A good soil can be ruined by improper construction practices. Perhaps the worst offense is to dig or work in the soil when it is too wet. Such practice results in the soil surfaces becoming smeared and effectively sealed before sewage is ever applied to the system. Another common fault in construction is the failure to set the distribution box properly, thus permitting most of the liquid to drain to one trench. Effects of this malpractice are especially severe in fields located on sloping ground where there is no opportunity for the overloaded trench to relieve itself other than through surface seepage. Care should be exercised during construction to prevent surface water carrying with it a load of silt from running into a partially completed field. Roof drainage or other storm water drainage is sometimes permitted to run over the surface of the disposal field, flooding it and producing wet weather failures. The list of construction

deficiencies is long but education and careful inspection have improved construction practice in many localities.

Even with good soil, design, and construction, the septic tank systems will fail unless the owner exercises reasonable care in operating the system. By failing to repair leaky fixtures or by using his home for commercial purposes the owner can overload the system and cause it to fail. The tank should be inspected annually and cleaned before sludge is carried to the disposal field and damages the soil. Engineers recommending septic tank systems for sewage disposal should make sure that all parties concerned are appraised of these limitations.

The septic tank is a poor temporary or interim treatment facility. In many instances, septic tanks have been permitted in areas where due to poor soil conditions there was little, if any, chance for continued successful operation, on the assumption that public sewers would be provided as soon as the population density reached the point where such service would be practical. Although such rationalization may be sound in theory, there are many practical reasons that work against such a plan. Some of these reasons are:

(a) Not all systems fail at the same time and it is difficult to get home owners who are not having trouble or who have recently spent sizeable sums to repair their systems to support community action that will cost them more money.
(b) Public resistance to bond issues and higher taxes. In addition to the expense, the inconvenience entailed in digging up streets and lawns tends to delay action until the situation can be extremely critical.
(c) Many home owners, fearing the adverse effect on property values, refuse to admit that failures are occurring, and do not support remedial action on a community basis. In addition to these negative reasons for not installing septic tanks as interim treatment facilities, there are several positive reasons why septic tanks should not be used. These are:

    (1) Any failure during the interim period creates a serious public health hazard.
    (2) The cost of installing sewers prior to construction of houses and streets is appreciably less than the cost of similar installations after the houses and streets are constructed.

A strong, well-organized health department is needed to prevent abuses with the septic tank system. A good ordinance defines the conditions under which septic tanks can be used. It requires enough useable area on each lot for two complete disposal fields, empowers the proper authority to establish minimum requirements for both septic tank and disposal field, requires an approved permit before construction starts, requires individual design and construction by competent, responsible, authorized parties, and requires a final inspection of the system before it is covered.

Community Systems.—Privately owned community systems refer to those systems constructed generally by a housing subdivision developer. Maintenance and operation are provided in one of a number of ways, depending on local conditions.

In many states, requirements for use of general obligation bonds are so restrictive that public financing of sewer systems for fringe area developments is most difficult. Greenleaf[3] reports that such a situation exists in

Florida where state authorities do not approve developments unless the subdivider provides both public sewer and public water supply systems. It provides for review of proposed subdivisions in unsewered areas by the health agency before the subdivision is recorded and lots are sold.

The use of public credit in Missouri is also difficult. The state constitution requires a two-thirds majority vote of approval for issuance of general obligation bonds and a four-sevenths majority for issuance of revenue bonds. An interesting arrangement is used by a developer in the Kansas City area. He installed the sewers and built the treatment plant for Leawood which now serves approximately 3500 homes. Costs of construction was included in sale price of the houses. Costs of operation are assessed and collected through the Homes Association. In order to prevent the construction of treatment plants in undesirable locations, this private corporation has recently negotiated contracts with developers of land upstream to provide sewer service, exclusive of laterals which the developer must build for a fee of $400 per house plus maintenance charges.

In nearby Jackson County, Missouri, the general practice is to require the developer to provide sewers and a temporary treatment plant which the county will operate until a main or interceptor sewer connection is available. A $200 escrow payment per house must also be deposited with the county. This money will be used to pay for trunk sewers and additional treatment facilities as development requires. No public funds are involved.

In New Jersey, all privately owned sewer systems operate under the supervision of the Department of Public Utilities and rates charged for services must be approved by the department.

Lee[4] reports about 80 privately financed and operated districts in Florida. Trust deed procedures help insure continued proper operation and maintenance. He also pioneered in "stage development" of such systems with only portions of the sewerage systems and units of the treatment facility being built as homes were constructed. He fully favors such a procedure over individual septic tank systems.

According to Salvato[5] the total cost per home for sewerage, including treatment, should be approximately $700 to $900, depending upon lot width, degree of treatment required, and prevailing construction costs.

Experience in Kansas and elsewhere indicates that sewer construction costs in built-up areas are 2 to 3 times higher than in similar undeveloped areas. If sewers are provided as a part of the original development, the burden of the householder is eased because the initial sewer costs are included as a part of the original house loan or will be included in the taxes as special assessments. On the basis of discussions with F.H.A. officials, it appears that the appraisal system will, to a certain degree, reflect a higher appraisal value for houses in sewered areas, this factor is not sufficiently appreciated or recognized by the general public to affect the selling price of a house.

The financial advantage to the ultimate home owner of building a home with installed public sewers is indicated by the following data based on experience in the Los Angeles area:

AVERAGE COST OF SEWERS AND PRIVATE SEWAGE DISPOSAL SYSTEMS IN LOS ANGELES[6]

	Contractors					Average
	#1	#2	#3	#4	#5	
Cost of average system (assume average level land and 900 gal. tank)	$350.	$315.	$300.	$300.	$315.	$315.
Cost of average single cesspool	235.	185.	200.	200.	165.	200.
Cost of abandoning cesspool and connecting to sewer	110.	150.	180.	125.	110.	125.
Cost per lineal foot of sewer laterals by subdivider prior to paving	5.00/ft	--	4.50/ft	--	4.00/ft	4.50/ft
Cost per lineal foot by special assessment district after streets are paved						14.50/ft

The amount a home owner could afford to pay toward sewer outlet facilities under Los Angeles price conditions can be calculated from the above figures.

Cost of private system	$315.
Cost of abandonment of system	125.
Cost of sewer lateral when sewer is ultimately installed in paved street (1/2 of 60' of sewer) 14.50 x 30	435.
Total cost	$875.
Cost of sewers before paving 30 x 4.50	135.
Approximate saving by initially installing sewers = $875 - $135	740. per lot

At 4 lots per acre the developer and ultimate owners could afford to pay $740. x 4 = $2,960. per acre for outlet sewer facilities.

The savings will not always be this great, but engineers might well apply the long-range approach in showing local groups why it is important to look beyond the first cost in making an economic appraisal. Proper consideration of these factors by governmental and private lending agencies could result in a more liberal policy in approving loans and credits for sewerage systems to serve new developments.

According to Richards,[7] financing of community systems by large developers is being widely practiced in Florida, Pennsylvania, New Jersey, New

York, Colorado, California, Texas, Oklahoma, Kansas, Indiana, and Michigan.

Friel[8] reported in 1947 that Pennsylvania had more privately owned sewer systems than any state. One of the distinct disadvantages of this arrangement is the inability of the private company to compel property owners to connect to the sewer. However, this particular objection is not of serious consequence in the case of new housing developments where the developer connects the house to the sewer during construction.

The significant advantages in having developers build and pay for the sewage works are: (1) The developer can construct the sewers when and where he needs them, while public financed sewers frequently require months of planning. From two to five years are frequently required to organize, finance, and construct a sewer system for a large public district, (2) the public financing capacities are conserved for other community needs, and (3) the premature development of subdivisions is minimized.

The principal objections to private construction of sewers are: (1) The system is seldom designed to serve more than a small portion of a watershed. Developers are reluctant to provide sewer capacities to serve other properties in the watershed whose development is uncertain. (2) Adequate provision is seldom made for perpetual and effective operation of the system. Unless profitable operation can be assured, most developers want to be free of operation responsibilities. To overcome this, developers may create a sewer corporation composed of home owners to operate the plant and levy taxes. Such arrangements are satisfactory if the development is sufficiently large to provide the funds for employment of competent personnel.

In spite of its shortcomings, private financing of the sewerage utility is being widely used in many areas and should not be overlooked where the use of public districts and public financing is not feasible.

Publicly Financed Systems

A wide variety of administrative and financial mechanisms have been employed in metropolitan areas to provide satisfactory sewerage service for outlying areas. The most common include municipal annexation, sanitary districts, conservancy districts, federation and city-county consolidation, and intercounty-city contracts.

Annexation.—Annexation of contiguous fringe areas prior to development by the city and the extension of the city sewers to serve the area is an excellent method of providing sewerage service. This concept of annexation implies central city control and financing of sewer installations in fringe areas. Subdivision regulations and city-county planning authority in fringe areas have done much to permit orderly development of municipal services.

The practice of many cities of extending city water and electric services to unincorporated areas around the city without any plan for providing essential sewerage service has frequently resulted in ill will between the city and the fringe area and has retarded annexation. Provision of selective or partial service, such as sewerage but not water, to contiguous developments will also create difficult collection and administrative problems where service charges are used. In many unincorporated areas, the collection of sewer services and charges would be difficult because generally punitive closure of sewers is not feasible for default of payment. This situation is being experienced in many southwestern areas where unincorporated developments have their own water systems but desire to utilize the sewerage system of the central city.

In some cases the central city must choose between having partially treated sewage from contiguous areas flow through the city with the hazard of occasional accidents, or permitting the sewage to be discharged into the city system.

From the standpoint of local administrative control, annexation is a very desirable method as it permits cities to extend their zoning and planning regulations into the fringe area early enough to prevent the establishment of undesirable housing developments.

The ability of cities to annex is seriously hampered in many states by the complex and cumbersome requirements imposed by constitutions and statutes. For example, in Kansas, cities of the first class cannot annex unplatted tracts of more than 20 acres without consent of the county officials.

Sanitary Districts.—A sanitary district was defined in 1945 by a committee of the Sanitary Engineering Division of the Society[9] as "a local governmental unit created under a state-enabling act to finance, construct, and operate works for the collection and treatment of sewage, including industrial wastes, amenable to joint treatment, and to provide adequate final disposal of the effluent and by-products of the treatment processes".

According to the above committee, districts are usually created by: (1) Petition of qualified voters, (2) by action of governing bodies, and (3) by action of the state, through such agencies as the State Health Department. Creation of districts by petition was favored.

The committee further reported that a district may encompass any of the following: Part of a municipality, an entire municipality, a single municipality plus some outside territory, a group of municipalities with some unincorporated territory or area in a county, a single county or area within a county, an area with an arbitrary boundary line, or a river valley or drainage basin.

The advantages of handling sewage, water, traffic, and numerous other services on a metropolitan basis are generally recognized. The difficulty is in accomplishment. From a review of the history of the existing metropolitan organizations now handling water and sewage, it appears that very few of them came into being until the cluster of urban communities were faced with a water or pollution situation of such magnitude that they could not solve it individually or through informal intercity cooperation. Though lacking the flexibility of integrated systems, special districts do provide the taxpayer with a direct financial accounting mechanism for the function being performed.

Sanitary districts are created, primarily, to accomplish objectives which cannot be attained through normal governmental agencies. Some of the reasons for creating a district may be:

1. To provide services to two or more political subdivisions.
2. To provide facilities and services for only part of a political subdivision and require those using the service to pay for it.
3. To expand the normal, legal debt limit.

A tabulation in American City in 1945 showed that 37 states had laws which authorized formation of sanitation or sewerage districts,[10] and a great growth has occurred since that time. The states with the largest number of such districts were:

California	180
Ohio	167
Washington	146
New York	Many

Wide variations exist in the sanitary district organization, and in planning, operating, and financing procedures.

Conservancy Districts.—Conservancy districts are generally created to provide a broad range of integrated services such as flood control, water supply, and sewage disposal. They have all of the desirable features of metropolitan sanitary districts and are generally organized on a watershed basis. The states of Ohio and Illinois have conservancy acts, the Ohio Law dates from 1910.(11) Under the Ohio laws, creation and operation of the conservancy districts are vested in the courts of common pleas. This type of jurisdiction is extremely popular because it makes possible centralization of authority and establishment of uniform rates. In Ohio, both Cleveland and Cincinnati are studying the possible use of this type of district.

In Illinois, the River Conservancy District Act has not been used to provide for sewerage service to date, but in an information sheet, "Sewer Financing", published by the Illinois State Department of Health, its use is highly recommended with a pertinent notation that the Conservancy District has no relationship to political boundaries. The Conservancy District Law as passed in Indiana in 1947 was found unconstitutional.

Multi-purpose districts are considered more desirable by the political scientists and have special single purpose districts as they simplify local government.

Federation and City-County Consolidation.—Federation, which has yet to be employed in the United States, has been used very successfully in Toronto, Canada. Toronto, a city of 1,250,000 population, had thirteen municipalities before federation. Under the Toronto Metropolitan Act, the local city authority manages strictly local matters, while certain metropolitan functions and services were transferred to a metropolitan authority. These services include water, sewage, housing, education, arterial highways, metropolitan parks, welfare agencies, and over-all planning. In the sewage field, the authority assumes ownership of the sewage treatment plants and the trunk sewers, and it accepts sewage from thirteen municipalities through meters. The authority bills participating cities for the service rendered, and they in turn collect the bills from their citizens. The authority is governed by a 24-man council composed of representatives from the municipalities. The Province of Ontario grants $4.00 per capita aid to the authority each year. Dade County, Miami, Florida has studied federation and plans a modified form of city-county consolidation.

Intercounty-City Contract.—The intercounty-city contract accomplishes the same thing as city-county merger through a series of long term contracts between the various political subdivisions involved. The service is generally provided by the central city. For example, in the Detroit metropolitan area, there are two such projects: (1) The Oakland project which is founded on a series of long term contracts, one between the city of Detroit and Oakland County, and the other between Oakland County and subdivisions, with each subdivision being permitted to decide how it wishes to raise the money to pay its contract; and (2) the Macomb-Wayne project, which differs from the Oakland project in that Wayne County appears as a party between Macomb County and the city of Detroit.(12) The project is thus founded on a city of Detroit-Wayne County contract, and a contract between Wayne and Macomb counties. There is also a series of contracts between Macomb County and its subdivisions. Under the terms of the Oakland County contract, Detroit provides Oakland County with 30 cfs sewer capacity for $137,883.60 per year, and provides

additional flowage rights up to 50 cfs for a surcharge of $4,596.12 per cfs per year. The sewage treatment charge is based on water consumption in Oakland and is set at $28.90 per mg of metered water. The charges in the Macomb project are similar, and the estimated annual household charge is $5.00.

City-county agreements, renewable every five years, are also used in the Atlanta-Fulton-DeKalb County area of Georgia.

The Highland Park area adjacent to Topeka, Kansas, contracted with the city for treatment of its sewage for a number of years prior to annexation.

Los Angeles City has permitted sewage from Burbank, Glendale, Santa Monica and Beverly Hills and other surrounding communities to be discharged into the Los Angeles system under contract for years.

There are uncertainties in arrangements based on intercity-county contracts, primarily questions of workability and permanence; but the fact that they are working satisfactorily in the above and other areas makes them worthy of consideration.

Financing Methods

According to Pasarew,[13] financing of sewers may be by:

General obligation bonds
Revenue bonds
Special assessment
Revolving funds
Direct financing by the subdivider

He also stated, "The size of the suburban areas is not a major consideration in the organization of a sanitary district".

While most districts secure needed revenue from ordinary ad valorem taxes, assessment charges, service charges, or a combination of these, and many special procedures are in use.

The Baltimore County Metropolitan District requires that the developer of a new tract deposit the full cost of utility extensions before houses are built. Later he is refunded to the extent that the extended system becomes self-supporting by front-foot assessments ranging from $0.17 to $0.33 per year. A second method is based on a charge of $4.00 per front-foot for each property owner's half of the cost of local sewers, plus the service charges, and the ad valorem taxes on the district.

There is a growing interest in revolving funds, sewer trust funds, escrow funds, and grants or some type of tax relief until an area is well developed. The escrow fund method is used in Jackson County, Missouri. The developer is required to pay a special amount into an escrow fund when the houses are built. The funds are used to help pay for the trunk sewers and permanent treatment facilities which will be constructed later. This revolving fund can be used to finance early construction of sewers in partially developed areas, which is the method being proposed for water systems in Oklahoma. The aforementioned Los Angeles acreage charge is another financing method.

Oregon statutes permit counties to create revolving funds to provide money to finance the construction of sewage works in unincorporated areas provided such construction is in accordance with a legally adopted master plan. A levy of two mills for a period of five years is authorized for the creation of the fund. All funds loaned are without interest and may be repaid over a period not to exceed 30 years.

Direct grants from the federal government, state legislature or central city provide another possibility for financing assistance. The 1954 Federal Housing Act provides for planning grants-in-aid to state or municipal planning agencies. Grants for master planning are made to these agencies on a 50-50 matching basis. Community facilities planning advances and other special provisions for federal aid are applicable in certain cases.

Public Law 660 of the 85th Congress provides for construction grants to municipalities up to $250,000 or 30% of the cost of the sewage treatment works. These funds are not available for the construction of sewers.

Kansas City, Missouri, at one time considered the feasibility of reducing the taxes in newly annexed areas where only partial city services are provided. It was found that citizens in the newly annexed areas were demanding full services for full taxes.

GENERAL CONCLUSIONS

The acceptability of individual sewage disposal systems depends upon soil, ground water, topography, lot size, and land use. Individual systems, properly designed for single-family homes, are functioning satisfactorily in some places, but even under favorable soil conditions, commercial, multiple family, industrial, and some individual systems do fail and will cause nuisances or health hazards unless promptly corrected.

The acceptance of individual systems should be subject to review and approval by qualified personnel working under the direction of experienced sanitary engineers. The review must be made before subdivisions are approved. In this review, the question of availability of sewers is an important factor. It must be kept in mind that it is not economically feasible to develop some areas for housing and at the same time provide full protection to the health of the people. However, if sewers cannot be economically provided, the reviewing authority might be guided by Hilbert[14] who said, "The impression that progress is inevitable and that certain areas have to be developed is a figment of the imagination".

Community systems may produce serious problems unless they are properly designed and adequate provision is made for maintenance, expansion and operation. If a large number of small community systems are installed, high operational costs may result in insufficient attention and undesirable conditions.

The development of a master sewerage plan for a metropolitan area on a watershed drainage basin is therefore an essential first step in orderly land development. Even with a master plan and a system of trunk sewers it is essential that practical ways and means be provided whereby developers with economically sound developments can obtain outlet sewer facilities for their property. On the other hand, no public agency should be required to build sewers for a project that is not economically sound. Trust or escrow funds with acreage, area, or front-foot charges and assessments are important steps in helping to solve the problem. Under proper conditions, consideration may be given to the use of dry sewers for purposes of economy. It is very important that public officials, builders, developers, and citizens face squarely the problem of uncontrolled development and adopt measures necessary for sound planning and provision of sanitary services. Leadership in this educational effort should be assumed by engineers individually and as a profession.

Unfortunately, each professional specialist or expert regards himself as possessing the broad view while others are narrow. Actually, no profession possesses enough experience or insight to understand all that is involved; consequently, professional team work is essential.

A large variety of administrative relationships among communities in the metropolitan areas have been used in solving sewerage problems in fringe areas. These have needed tailoring to the laws and physical conditions of the particular area.

Respectfully submitted,

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APPENDIX A.—EXPERIENCE IN LOS ANGELES, CALIFORNIA

Prior to 1945, when subdivisions were springing up in the then agricultural San Fernando Valley portion of the City of Los Angeles, there was no review of the sewage disposal requirements prior to the approval of subdivisions. With the initiation of a sanitary engineering program in 1943, the Health Department promoted and the City Council adopted a modern code requiring adequate private sewage disposal systems.

While this was an important measure, it was not a solution to the problem. Lots were purchased by individuals and homes were partly constructed before the owners learned that soil and ground water conditions were not suitable for private sewage disposal systems. As a first step in preventing construction of housing in areas which were unsuitable for private sewage disposal systems, the Health Department prepared maps of the unsewered areas, showing as precautionary areas those unsewered, undeveloped areas where soil topography and ground water conditions require careful, detailed investigation before private disposal systems could be approved. The Building and Safety Department withheld building permits in critical areas until the Health Department made a special review.

It soon became apparent that controls had to be applied before individual lots were developed. To correct this situation, the City Council amended the subdivision act to require Health Department review of all subdivisions with its report going to the Planning Commission.

Health Department recommendations for disapproval of many private

disposal systems resulted in demands for public sewer works. However, because no established method of financing connecting sewers was available, it was necessary for subdividers either to wait until sewers were extended by natural growth, or to provide a connection to the city sewer at their own expense and according to city specifications, with no assurance of any future reimbursement.

The next step in the evolutionary process was development of a reimbursement law which it was hoped would provide an equitable solution to this problem. Subdividers who installed sewers beyond the limits of their developments were presumably eligible to be reimbursed for excess size, depth and length of sewer. Financing and legal difficulties arose to make this procedure unworkable.

Acreage Charge Ordinance

Recently an acreage charge ordinance was approved by the City Council in a new move to solve this problem. This contemplates two types of systems, one in which tract sewers will be connected to the city sewerage systems before lots are developed. The second applies in areas where temporary, private sewage disposal must be used in locations approved by the Health Department until sewers are provided. In either case, the subdivider must install tract sewers as a condition of tract approval. By accepting the acreage charge, the city guarantees to extend the municipal sewer to the subdivision, the date when the outlet facility is to be made available depending upon need and other factors.

The acreage fee ordinance is actually a somewhat unusual concept. A determination is made by the city engineer concerning whether it is economically sound for the city to assume the obligation to provide sewers for the drainage area in which the subdivision is located. If it is determined that natural growth in the particular section will be such that the total acreage fees collected will pay for the sewer in the reasonably near future, the fee is accepted. When the conditions are suitable for private systems and a connection cannot be provided before the buildings are erected, the subdivider installs dry sewers planned for connection with the future sewer. The Health Department determines the type of temporary disposal system to be installed on each lot. These systems vary from a single cesspool in areas where the soil is coarse sand and gravel, to sections where the ground water is high, and in locations where the soil is relatively impervious, no private system is allowed, and the city's decision regarding acceptance of the acreage charge is based on whether it is financially sound for the city to provide the sewer connection immediately.

Local Assessment or Improvement Districts

Such districts are usually created for installation of local sewers. The districts are normally financed by contractors who have determined that property values are high enough to insure repayment of the cost of the sewers. The laws usually require approval of owners of over 50% of the benefited properties. The problem is complicated because few are willing to pay their share until the need for the sewer is obvious. An area in a proposed district may become partly or fully developed before property owners take the initiative of circulating petitions. Too often the first families that experience overflowing or backing up into fixtures find their neighbors whose private

systems are functioning satisfactorily unwilling to share the cost and to sign the petition.

Section 2808 of the California Streets and Highways Code provides a workable procedure for installation of such sewers by front-foot assessment procedures without a majority petition and without the usual delaying legal formalities. The Health Officer may certify that nuisances and health hazards result from lack of sewers. The establishment of the district may then be approved by a 3/4 majority of the governing body of the city or county, without a petition from property owners. This system is frequently used and materially reduces the extent and duration of the emergency which otherwise must precede remedial measures.

Administration of Districts

Districts are usually governed by a board of trustees or a group of appointed or elected officials. Districts of any size require proper, full-time technical and administrative direction. In some cases each community served by a district is represented on the district's governing body. Los Angeles County now contains 51 incorporated cities, plus numerous large urban districts in unincorporated territory. To have a single district handle the whole county would legally necessitate an unwieldy governing body. There would be little of common interest in many of the meetings of the group. These features have resulted in the formation of several separate and practically autonomous districts. In a report, "Sanitation and Health - Metropolitan Los Angeles,"(3) 13 districts in Los Angeles County with from 3 to 10 members on each of their Board of Directors are listed. There are now 19 districts.

Since 1928, the various county districts have jointly employed a Chief Engineer to head the central staff and, in fact, to direct the work of each district. This procedure provides the advantages of a central planning, technical, administrative, and operating organization, and in addition, permits local determination of program.

It is deemed of considerable importance that a centralized district administration of competent, full-time staff be used in areas large enough to provide such staffs by joint or single action. A sewerage survey of Orange County(1) (adjacent to Los Angeles County) recommended 10 districts with a central administrative organization.

A report, "The Collection, Treatment and Disposal of the Sewage of San Diego County, California,"(2) illustrates how a long range plan can be developed for providing sewerage and sewage treatment service in a relatively large county where conditions require the use of several independent systems and separate treatment and outfall facilities. The engineering study was recommended by a Metropolitan Sewerage Planning Committee which was born of suggestions by the Regional Water Pollution Control officer who stimulated action by consulting with the 5 cities comprising metropolitan San Diego and with the County Board of Supervisors. A Board of Engineers was appointed by

1. Bacon, V. W. "Sewerage Survey Develop a 40 Year Regional Plan". American City, April, 1948.
2. Rawn, A. M., Hyde, G. G., and Caldwell, D. H. "The Collection, Treatment and Disposal of Sewage of San Diego County."
3. Couch, W. W. "Sanitation and Health - Metropolitan Los Angeles." The Haynes Foundation, Los Angeles, 1952.

the County Board of Supervisors for "the development of a comprehensive master plan or program of sewerage, sewage treatment and sewage disposal for the entire county." The broad studies by the Board of Engineers resulted in an extensive plan for mains, interceptors and treatment works. However, it did not include a specific solution to the problem of providing outlet sewerage facilities for subdivisions located a considerable distance from the interceptors. The report's recommendation have not yet been adopted for action.

Paper 1614 April, 1958 SA 2

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Note: Paper 1614 is part of the copyrighted Journal of the Sanitary Engineering Division Proceedings of the American Society of Civil Engineers, Vol. 84, SA 2, April, 1958.

DESIGN OF VENTURI FLUMES AND CIRCULAR CONDUITS[a]

Closure by Edwin A. Wells, Jr. and Harold B. Gotaas

EDWIN A. WELLS, Jr.,[1] A.M. ASCE and HAROLD B. GOTAAS,[2] M. ASCE. —The venturi flume at the North Point sewage treatment plant in San Francisco described by Mr. Fraschina is one of the largest Palmer-Bowlus flumes. The authors had the opportunity of visiting this installation while it was under construction. It is significant that the methods employed in the construction of the flume were practically precise and that the flume design conformed, in general, with the findings of the laboratory studies which were underway at that time.

Mr. Fraschina has made a very able study of the behavior and accuracy of this flume and it should be noted from his article that the performance of a flume of this size conforms closely with the results of the laboratory tests. As Mr. Fraschina pointed out, the fact that the upstream channel is not circular has no relative bearing on the desirability or usefulness of the Palmer-Bowlus type of venturi flume for measuring flow.

Except for the minor variations mentioned by Mr. Fraschina, the North Point Palmer-Bowlus flume meets the general criteria which have been found applicable to these flumes. It should be especially noted that the energy criterion developed in the laboratory studies is well satisfied by the nature of the installation and its relative position to the hydraulic grade line through the sewage treatment plant.

The correlative tests that have been performed by Mr. Fraschina are the first large scale tests of this type to come to the attention of the writers and as such, provide a valuable background for the future utility of the Palmer-Bowlus flume. It is important to note that the dimensions of each Palmer-Bowlus flume installation can be measured after the flume has been built and the rating curves of the flume can be modified to reflect even very minor variations in the final construction. Because of the "standard" nature of the Parshall flume, rating curves for those flumes cannot be modified to reflect differences in the flume where the final flume as built differs from the flume as originally designed.

It has been brought to the authors' attention that several Palmer-Bowlus flume installations (in sewers) do not perform the metering functions satisfactorily. Very probably, this can be traced to the fact that prior to the availability of the energy criterion presented from laboratory tests, the Palmer-Bowlus flume installation was designed principally on the basis of dimensional variables and approach velocity.

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a. Proc. Paper 928, April, 1956, by Edwin A. Wells, Jr.,and Harold B. Gotaas.

1. Munic. Financing Cons., Stone & Youngberg, San Francisco, Calif.
2. Dean, The Technological Inst., Northwestern Univ. Evanston, Ill.

In the considered opinion of the authors, the most valuable product of the laboratory research is the development of a working criterion for the design of the Palmer-Bowlus flume based on its energy relationships. Since many times venturi type measuring flumes must be installed at points where rapid flow normally occurs in sewers or in treatment plant channels, an understanding of the energy limits within which a venturi flume can be used should increase the utility of such flumes. For conservative design, these flumes have heretofore been largely restricted to uses where tranquil flow can be assured. The Parshall flume, which was originally developed solely for use in earthen irrigation ditches where tranquil flow pertains, has been unwittingly used for a variety of types of installations where rapid flows are encountered. To the authors' knowledge, no energy criterion similar to that now available for Palmer-Bowlus flumes has ever been developed for the Parshall flume.

With its many outstanding features, the Palmer-Bowlus type of flume is becoming more widely used in the western United States. For normal installations, the design has not proved difficult or cumersome. However, because "standard" Palmer-Bowlus flumes have not been developed to the point where they are readily available in engineering handbooks, the flume type lacks the popularity and understanding attached to the Parshall flume. Obviously, there is a need for the further development of standard dimensions and rating curves for Palmer-Bowlus flumes to handle the normal metering problems such as those suggested by Ludwig and Ludwig.(10)

DISPERSAL OF POLLUTION BY TIDAL MOVEMENTS[a]

Discussion by D. I. H. Barr

D. I. H. BARR,[1] A.M. ASCE.—The Author has in general clearly stated the relationship between tidal movements and stream flow but has perhaps over simplified certain aspects which are of importance in the mixing of non-homogeneous water. In a stream of homogeneous water, of depth moderate to the travel lengths involved, certain water particles rather than portions of water will travel farther than others; i.e. if water of recognisable characteristics but of the same density as a stream of water with uniform flow be introduced at the surface at some point, it would be expected that, after an interval of time, some traces of the introduced water would be discernable at a farthest distance (over a short interval of time sufficient to allow for the random characteristics of turbulent eddy dispersion) almost equal to the distance that would be travelled by an imaginary particle which had maintained the path of maximum average velocity available within the diverging dissipation zone appropriate to the point of introduction. Similarly the least distance at which traces would be discernable would be approximately equal to the distance travelled by a particle following the path of minimum average velocity available within the diverging dissipation zone. The furthest traces would be expected to be on a broad front slightly below the surface at the point of maximum velocity and the nearest traces to the point of entry either at the bottom in a shallow stream or at least considerably deeper than the furthest traces. Between the two the distribution would be complex with certain configuration based characteristics and certain random characteristics. In tidal streams the picture is complicated by the cyclic nature of the flow and in very many practical examples, both tidal and non-tidal, by either thermal or, in lower tidal reaches, saline statification which can be regarded as influencing the shape of the diverging dissipation zones which are inherent in moving water. If introduced water is of a different density to a uniform density stream, it will tend initially to the top or to the bottom, i.e. its particular dissipation zone might be considerably modified. If, in addition, the stream is itself statified the introduced water may be of intermediate density and thus might also tend to its own density in the statification. Such initial positioning would be even more the case at high or low slack water in tidal rivers. Has the Author observed differences in dispersion rates due to differing density differences, both in the river and between the river and the inflow. Although not applicable to the cases quoted, the presence of both saline and temperature variations can have considerable influence on dissipation and travel of effluent.

a. Proc. Paper 1408, October, 1957, by Thomas M. Niles.
1. Glasgow, Scotland.

The study of sewage dispersal, saline intrusion and mixing, industrial effluent dispersal including radio-active effluents, the flows and currents of tidal and non-tidal rivers as they affect mixing, the dissipation of surplus heat and the consideration of industrialised rivers as biological environments, and particularly as short-term environments of migratory fish which may or may not kill the fish, should not be regarded as separate subjects but as branches of the same subject in which specialists in considerably differing aspects of civil engineering and science have a common interest and where each group can learn from the observations and deductions of the others.

LIQUID CONTENT OF GARBAGE AND REFUSE[a]

Discussion by P. R. Krige and G. H. Teletzke
Closure by John S. Wiley

P. R. KRIGE,[1] —The paper has undoubtedly thrown the question of moisture content of raw organic wastes immediately prior to composting into a new perspective. Hitherto it has been generally agreed that a knowledge of the moisture content of the raw materials was sufficient to determine the proportions in which components should be blended in order that optimum moisture conditions prevail during humification. Workers in this field of sanitary engineering have been aware of the need to distinguish between superficial moisture - that which temporarily adheres to the surface of waste materials as a result of rain or otherwise - and inherent or combined moisture, which is a natural ingredient of the substance. Superficial moisture is rapidly reduced by draining or exposure in dry weather, but inherent moisture requires prolonged desiccation. Obviously, once the raw material is mechanically disintegrated or comminuted, these two moistures are no longer distinguishable.

The author of the paper has very conclusively shown that a reconsideration of our ideas on moisture content is necessary and that it would be more correct to include all the natural greases, fats, oils and waxes, as well as the true moisture, in the general designation "liquid content." If the lipids are not already fluid at normal atmospheric temperatures, it is reasonably safe to assume that they will be at the thermophilic temperatures of an active compost system and it is then that they either replace the air from the minute pores or interstices of the materials, or prevent air from entering.

Although the writer of this brief discussion is prepared to endorse the author's views regarding the possible detrimental effect of lipids on aeration at elevated temperatures, the opinion is ventured that the author has not weighed the decided advantage of lipids as the most acceptable and readily available form of carbon to compost organisms, against the possible disadvantage of air exclusion. It is generally known that all lipids are completely removed by composting, which is an indication that these substances are preferentially utilized by compost organisms as carbon sources, even to the extent of complete oxidation, as against vegetative forms of carbon which convert to humus (compost) with a residual carbon complex.

In this connection, a brief reference to an investigation conducted by the writer in 1951 will not be out of place. Because of the fact that the methane fermentation process (anaerobic digestion), as employed at most sewage

a. Proc. Paper 1411, October, 1957, by John S. Wiley.

1. Water Research Div., National Chem. Research Lab., South African Council for Scientific and Industrial Research, Pretoria, South Africa.

treatment plants for the biological treatment of sewage sludge, does not change the quantity and character of the ether-soluble fraction of the sludge - a study was made to determine the possibility of an aerobic breakdown of these fats which are undesirable in the digested sludge when used agriculturally. To accentuate the fat factor, floating scum was skimmed off the sedimentation tanks receiving raw sewage and this greasy material was placed as several one-inch layers in an experimental compost heap containing also other ingredients to give a reasonable C/N ratio. These grease layers initially gave a 50% ether-soluble figure, being mostly kitchen fats. After three months undisturbed composting, the heap was opened and what remained of the grease layers gave a 5.0% ether-soluble figure; on examination, it proved to be unsaponifiable grease and oil (ex service stations!).

Academically speaking, the author's case in favour of incorporating lipids with the moisture under the heading "liquid content" is sound and should be adopted in research projects, but in practical municipal composting of heterogeneous household refuse, the composition of which will vary widely from day to day, the lipids figure may be altogether insignificant (cf. Kobe, Japan: author's paper). Furthermore, the determination of the ether-soluble fraction in coarse refuse - and for that matter the moisture content too - presents a problem in sample collection and preparation of no small magnitude. This problem is encountered in compost installations where crude refuse is not comminuted before composting.

Basically, the author is correct when he advocates that "liquid content" should be expressed as a percentage of the volatile (organic) constituent of garbage and not the original material which has a fair percentage of uncompostable, inorganic substances. This is even more so in the case of South Africa where true garbage is not known and our mixed refuse can have "ash" figures of 40 to 90% on the "as received" basis!

The author is to be congratulated on the thoroughness of his research.

G. H. TELETZKE.[2]—The determination of the concentration of liquids in compostable material as proposed in the Wiley paper seems to be logical and of more value than moisture content alone. The writer wishes there were more data to support the beliefs expressed.

Perhaps an even more reliable liquid concentration could be obtained by including only the liquid lipids in the calculation. Not all lipids are liquid at the composting temperature. The writer has no knowledge of the proportion of liquid to solid lipids in garbage. (Perhaps Mr. Wiley does.) The determination of liquid lipids would be more involved than total lipids but it could be done. Whether it would yield any more reliable or usable data is questionable.

JOHN S. WILEY,[3] M. ASCE.—Dr. Teletzke indicates that a proportion of the lipids in compostable material may not be in liquid form at the composting temperature whereas Mr. Krige states that "it is reasonably safe to assume that they will be (fluid) at the thermophilic temperatures of an active

2. Asst. Prof., Dept. of San. Eng., School of Civ. Eng., Purdue Univ., Lafayette, Ind.
3. Senior San. Engr., U. S. Dept. of Health, Education, and Welfare, Public Health Service, Bureau of State Services, Communicable Disease Center, Technical Development Labs., Savannah, Ga.

compost system." While temperatures of 168° F have been reached in the 15-gallon units at Savannah and other workers report maximum temperatures of 170 to 175° F, it appears that a readily attainable temperature of about 140° F (60° C) may be preferable than higher values. Preliminary work on optimum composting temperatures by the writer(1) indicated that better decomposition occurred at temperatures of 131-145° F than in similar units allowed to reach maxima of 153-162° F. Also, 140° F for several hours is a sufficiently high temperature to kill pathogenic organisms, flies in any stage, and weed seeds.

Lipids in garbage or refuse consist primarily of oils, fats, and, to a lesser extent, waxes. Distinction between oil and fat is purely a physical one, the former being liquid at ordinary temperatures, the latter being solid. They are mixtures of glyceryl esters of the fatty acids with a particular oil containing a number of such esters of various acids. Table I shows the important edible oils and fats which would likely be present in garbage or refuse and none of them have either melting or solidifying temperatures as high as 60° C. In most cases, the oil or fat as a whole has a lower melting point than its primary fatty acid component, if one can judge by comparison of melting and solidifying temperatures. It, therefore, appears that nearly all lipids expected in refuse would be in liquid form at active composting temperatures. It is granted that waxes would probably not be in liquid form, but practically none would be present in garbage and only small amounts in refuse.

Although lipids may hinder composting by deterring aeration, they are readily oxidized by the microorganisms in an active compost under aerobic thermophilic conditions, as Mr. Krige states. Lipids have over twice the food energy value of most other food wastes. In some measurements of heat production during composting,(1) the writer found that lipids decomposition accounted for about 75 percent of the total heat production. The writer has unpublished data on initial and final lipids contents and losses in weight, the averages for 25 composting units with sorted Savannah refuse being:

	Initial Dry Weight		Final Dry Weight		Loss in Weight
	%	Lb.	%	Lb.	%
Lipids content	10.9	1.77	3.7	0.43	75.7

It is realized that something must be known of the raw materials that are to be composted and that representative sampling and adequate analyses are difficult and time-consuming. In the United States, some of these difficulties will have to be overcome before composting can become an acceptable method of organic waste treatment. The proposed liquid content formula comprises three relatively simple analyses: moisture, lipids, and ash (or volatile solids). The tests are far simpler than those for carbon and nitrogen. The liquid content value should indicate whether the physical environment is proper for aerobic thermophilic composting of organic wastes. Other important physical conditions are particle size and structure. The liquid content may be easily adjusted by adding water if it is too low or by adding waste dry materials such as paper or sawdust if it is too high. Without proper physical conditions, the wastes will not compost properly even though

TABLE I

EDIBLE VEGETABLE AND ANIMAL OILS AND FATS[2][3][4]

Group	Common Name	Solidifying Point, °C.	Insoluble Fatty Acids	
			Melting Point, °C.	Solidifying Point, °C.
Olive Oil	Olive oil	+ 2 turbid, - 6 solid	26-30	16.9-26.4
	Almond oil	- 15 to - 20	13-14	9.5-11.8
	Peanut oil	3	---	30.5-39
Rape oil	Rape seed oil	- 10	18.5-20	11.7-13.6
Cotton-seed Oil	Cotton seed oil	+ 12 to - 13	34.5	32-35
	Sesame oil	- 4 to - 16	25-35	23-32
	Corn (maize) oil	- 10 to - 20	17-20	14-16
Linseed Oil	Poppy seed oil	- 16 to - 18	20.5	17-19
	Sunflower oil	- 17	22-24	18-19.8
Cocoa-butter	Cocoa butter fat	21.5-27.3	48-53	47.2-49.2
	Cotton seed stearin fat	16-22	27-30	35.1
Coconut Oil	Coconut fat	14-22	24-27	21.2-25.2
	Palm & palm kernel	20-27.4	25-50	20-45.5
Lard Oil	Lard oil	+ 4 to - 2	33-38.4	27-33
	Lard oil (fatty tissue)	27.1-29.9	37-46.6	36-42.4
Tallow	Butter fat	19-24.5	38-41	33-39
	Beef tallow (fat)	31-38	42.5-44	37.9-46.2
	Beef marrow (fat)	29-31	45-46	37.9-40
	Mutton tallow (fat)	36-41	33.5-49	40-48.5
	Soybean oil	- 10 to - 16	26.2-27.5	21.2

all of the essential foods, minerals, and minor ortrace elements are in proper balance.

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VORTEX FLOW THROUGH HORIZONTAL ORIFICES[a]

Discussion by John W. Cunningham

JOHN W. CUNNINGHAM,[1] M. ASCE.—This paper is a valuable contribution to our knowledge of hydraulic behavior, and has an important practical application in the design of regulators for intercepting flow from combined sewers. The design utilizing the vortex principle proved very successful for the diversions of the Portland intercepting sewer system.

The Portland sewer system was predominantly one of "combined" sewers, carrying during the rainy season large volumes of storm water. It had 61 trunk sewer outfalls, of which 57 were intercepted and brought to a single treatment plant. By reason of elevations and grades, it was in many cases necessary to make diversions from laterals which fed the trunk sewers, resulting in more than 150 separate points of diversion. Many of these diversions became flooded at high river stages, with the result that not only sanitary and storm flow but also backwater from the river might, in the absence of control, enter the interceptors. Altogether, the problem of diversion was complicated and difficult.

Consideration was given to all of the regulating devices that have been described in the literature, and a considerable correspondence brought out additional ideas. Models were made for a number of devices and some of these were given small scale operating tests, though not on a quantitative basis. Reliability under variable and adverse conditions was given prime consideration. The regulators had to operate with a minimum of maintenance in locations where access was difficult and where they probably would be neglected.

An orifice diversion of any kind, horizontal or vertical, is subject to the formula $q = Ca\sqrt{2gh}$. For a cylindrical pipe sewer, for flows up to two thirds or more of the depth, the flow varies approximately as the square of the depth. Therefore an orifice, and particularly a horizontal orifice, diverting from a sewer constitutes an excellent regulator and has been widely used in different combinations, sometimes with a float operated device to completely close the diversion when the sewage level reaches a given point.

Following the preliminary studies for the Portland system, which were made by the writer with the cooperation of Mr. Ray E. Koon, it was decided to use the horizontal orifice diversion in most cases, but for a few of the larger sewers to use a plug valve type of closure that would completely interrupt the flow during major storms and offset the excess coming from the simpler type of diversion. One worry related to the problem of grit. It was feared that an orifice located at the sewer invert would pick up most of the

a. Proc. Paper 1461, December, 1957, by J. C. Stevens and Richard C. Kolf.
1. Cons. Engr., Portland, Ore.

bed load grit and carry it into the interceptor system, a very undesirable condition. Designs were at this stage when Mr. Koon retired from activity in the project, and Mr. Stevens, his partner, succeeded him. Mr. Stevens had previous experience in hydraulic model studies, and it was decided to construct and test a model diversion quantitatively and primarily to observe the behavior of the bed load grit. The model was made from sheet metal, and the curved shape surrounding the orifice was for convenience in fabrication. When tested, the very desirable vortex action was observed. This action was entirely fortuitous and unexpected, and was an added advantage for a device which would have been practical in the absence of vortex action. In the same experiments was observed that the bed load grit was largely carried past the orifice and over the dam. The vortex seemed to skim off the top sewage carrying less grit. The device was adopted for practically all of the Portland diversions.

The writer also designed a plug valve closure, somewhat like that shown in Fig. 11 of the paper, but ruggedly built of cast iron to meet the severe conditions of a sewer manhole. A modification of this, made of sheet metal and wood, was installed in the 13th Avenue diversion but due to the flimsy construction did not prove successful. The idea has possibilities if the construction is sound and permanent.

The later model studies, conducted at the University of Wisconsin by Mr. Stevens and Mr. Kolf give further design data and coefficients which will permit engineers to plan horizontal orifices with assurance of the desired results. There may be other applications in addition to the one that has been demonstrated, and which is of prime interest to sanitary engineers.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203(SA2), 1204 (SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218(SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226(WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230 (HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246(HW2), 1247(HW2), 1248 (WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257 (IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267(PO3), 1268 (PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275(SA3), 1276(SA3), 1277 (HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283(HY3)[c], 1284(PO3), 1285(PO3), 1286 (PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297 (HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306 (SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315 (ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324 (AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347 (BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c],1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4),1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c]. 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c],1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

c. Discussion of several papers, grouped by divisions.

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